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2. A 50-word summary must accompany the paper.

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4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

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8. Papers should average about 12,000 words in length and must be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

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CONSTRUCTION DIVISION

Proceedings of the American Society of Civil Engineers

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FUNDAMENTALS OF ARCTIC BLASTING

By Clifton W. Livingston[1]

FOREWORD

This is the second paper of this nature. The September, 1959 issue of the Construction Division carried the paper "Ventilated Building Foundations in Greenland," by Roger H. Williams. In an effort to comply with the desires of the Executive Committee of the Construction Division, it is hoped to publish at least one paper per Journal on some phase of Adverse Weather Construction. In the not too distant future, this division hopes to hold a symposium on this important type of work. Capt. Palmer W. Roberts, USNCEC, Chairman of the Committee for Adverse Weather Construction, welcomes all papers on this subject.

SYNOPSIS

The behavior of materials in blasting may be classified into (1) the shock type, (2) the shear type, or (3) the viscous damping type depending upon physical properties of the material and the scale of the experiment. Blast effects are determined by the relation between the energy of the explosion and the mass of the material to which the energy is transferred.

INTRODUCTION

Experiments have been conducted during the past 10 yrs. by the Corps of Engineers, U. S. Army, on the effects of explosions in rocks, soils, frozen ground, water, ice, snow, and air. The experiments include both military and commercial explosives in charges of various shapes ranging in weight from a few ounces to several tons.

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 1, February, 1960.

[1] Barodynamics, Inc., Wheatridge, Colo.

This paper deals with the portion of the work done by Snow, Ice, and Permafrost Research Establishment, Corps of Engineers. The work began in 1952, at Houghton, Mich. in shallow frozen ground. It was continued the following year in deeply frozen ground at Fort Churchill, Manitoba, Canada. The field work for explosions in ice and snow followed, and took place on the Greenland Ice Cap. As the work progressed, various shock effects were measured. For example, the latest experiments, which were done in snow, included seismic measurements, measurements of under snow pressure, acceleration, and displacement; measurements of the time at which snow was broken from the surface above the charge and the velocity of "flyrock" travel. Measurements were also taken of shock pressure in the air produced by charges detonated below the surface, and of shock pressure in the snow produced by charges detonated in the air. The instrumentation phases of the test program were supported technically by Waterways Experiment Station, Corps of Engineers.

MECHANICS OF FAILURE IN BLASTING

Rather than attributing blasting action in various materials entirely to reflection of a shock wave from a free surface, the view held here is that the behavior of materials subject to dynamic loading may be classified into at least three different and distinct types depending on the physical properties of the material and the geometric scale of the experiment.

The three types are classified here as:

(1) The shock type;
(2) The shear type, and
(3) The viscous damping type.

In each type, a disturbance passes outwardly from the explosion cavity. If the disturbance travels at a velocity greater than the sonic velocity of the material it is known as a "shock wave". An increase in velocity above the sonic velocity is the result of an increase in the density of the material through which the disturbance travels. The pressure in the material rises abruptly at the shock front and decays approximately exponentially behind it. In compressible substances, the gage pressure declines to negative values before the material returns to its initial state. The magnitude of the peak pressure at the shock front depends on the physical properties of the material within the range of the experiment, the energy of the explosion, and the distance from the explosion to the point at which the peak pressure is measured.

If deformation is non-recoverable and has been accomplished without loss of cohesion, the material behaves plastically and the deformation is a "plastic deformation". If deformation is recoverable, it has been accomplished without loss of cohesion and is an "elastic deformation". If fracture occurs, cohesion is lost. In blasting, cohesion generally is lost at some stage. The terms "brittle-acting" and "plastic-acting" are used here to describe the initial behavior of the material. Rocks that are "brittle-acting" exhibit elastic behavior and store energy before failure occurs. Applied to the ideal case, a plastic substance is one that dissipates all of the induced energy by internal friction. So far as is known at present, ideal plastic behavior does not occur in blasting.

Most rocks when loaded as in a testing machine deform in part elastically and in part plastically. Although deviation from ideal elastic behavior is less in blasting than at rates of loading, such as in a testing machine, it nevertheless occurs. Accordingly the terms, "brittle-acting" and "plastic-acting" are used

here, not in the classical sense, but to differentiate between materials that deviate to a greater or to a lesser extent from ideal elastic behavior.

The behavior of rocks in blasting is influenced by geologic processes such as alteration, jointing and fracturing. The proportion of the total deformation that is non-recoverable is greater in rocks that have been altered by most types of alteration than in unaltered rocks. Pre-existing planes of weakness such as joints, bedding planes and faults, affect the attitude of fractures formed by blasting and modify the size and shape of the excavation. Because of the greater homogeneity of frozen ground, ice, and snow in comparison to soils and to rocks, evidence has been obtained on the mechanics of failure in blasting that otherwise would be difficult to observe.

The three types of behavior of materials subject to dynamic loading are illustrated in the subsequent photographs.

Shock type behavior is characteristic of brittle-acting solids and is a result of reflection of the shock wave from a free-face. The material fails in tension and the planes of failure are dish-shaped or approximately parallel to the free-face. Failure begins at the free-face and progresses in a series of stages back towards the explosion cavity.

Shear type behavior is characteristic of plastic-acting solids and is a result of expansion of the explosion cavity by compaction and plastic deformation. At a relatively short distance from the center of the explosion, the pressure in the medium behind the shock front is greater than the pressure at the shock front. As the explosion cavity expands, the material is displaced towards the free-face; and the magnitude of the displacement is related to the magnitude of the peak pressure. The outward displacement is accompanied by the doming and stretching of the free-face and by shearing failure, which begins at the explosion cavity and progresses outwardly into the material.

Viscous damping type behavior is characteristic of porous and permeable solids and is due in part to the elastic behavior of the solid and in part to the air in the voids. The shock wave that passes through a very porous, brittle-acting solid is damped rapidly, and is followed by a gradual rise in pressure to a peak several times larger than that at the shock front. Failure occurs in two phases: the first resembles the shearing type failure characteristic of plastic-acting substances; the second resembles the phenomenon of elastic rebound that is characteristic of brittle-acting substances and is a result of reversal in the direction of displacement after the top of the explosion cavity has failed in shear and the pressure in the medium exceeds the pressure within the explosion cavity.

Fig. 1 shows the results of a blast in granite. Tension fractures have formed parallel to the surface of the rock above the explosive charge. The type of fracturing is characteristic of the behavior of brittle-acting rocks and is caused by reflection of the shock wave from the surface. In classifying the rock of Fig. 1 as "brittle-acting" it may be inferred that it is capable of storing energy during the compression stage. In attributing failure to reflection of the "shock wave" it is inferred that a shock wave exists and that the disturbance strikes the reflecting surface at supersonic velocity. The term "supersonic zone" therefore may be used to describe the spherically-shaped volume of material within which the shock wave travels at supersonic velocity. Experience has shown that the limits of the supersonic zone depend upon the energy of the explosive charge, the physical properties of the material, and the geometric scale of the experiment.

Fig. 2 shows the results of a blast in ice. Fractures have formed on vertical radial planes that pass through the center of the charge and on inclined planes at right angles to the radial planes. The fracture pattern differs from that characteristic of brittle-acting rocks in that tension fractures parallel to the ground surface are subordinate to those shown.

Pressure-time measurements show that the form of the disturbance in close proximity to an explosion cavity in ice resembles that within the supersonic zone in brittle-acting rocks. The measurements also show that beyond the supersonic zone the disturbance changes so that the shock-pressure at the wave front is small compared to the peak that is reached as a result of a gradual rise in pressure behind the wave front.

The existence in most craters in ice of remnants of an enlarged cavity be-

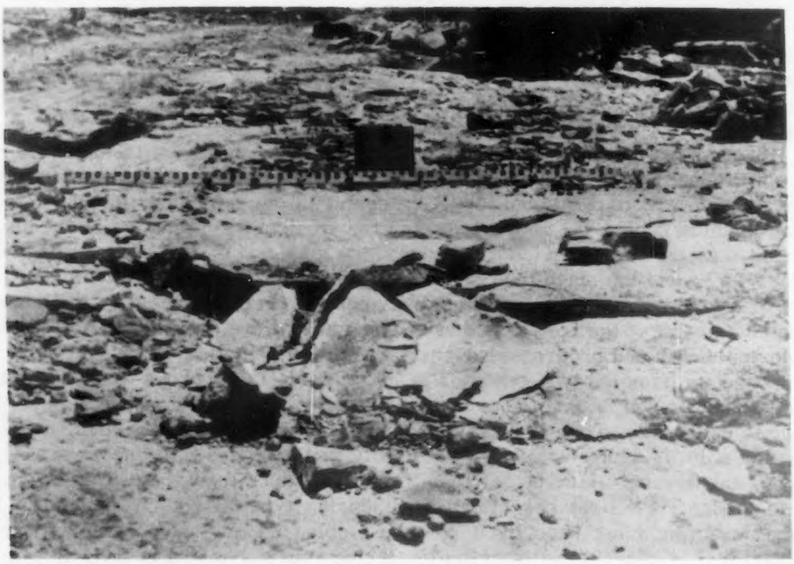

FIG. 1.—SLABBING IN GRANITE. SHOCK MECHANICS.

low the center of the charge, demonstrates that the ice surrounding the explosion cavity is displaced outwardly without fracture. Although deformation of the walls of the explosion cavity may not coincide with classical notions of plastic flow, the walls are hard and there is little evidence that cohesion has been lost. It is evident by inspection that deformation in close proximity to the explosion cavity is not recoverable. Accordingly, the deformation is classed here as a plastic deformation. The supersonic zone continues beyond the walls of the explosion cavity, but the shock wave is damped rapidly and reaches sonic velocity at a shorter distance than in brittle-acting rocks.

Fig. 3 is a photograph that shows the results of a blast in 0.45 density snow. Shock effects are secondary to effects associated with the rise and decline of pressure behind the shock front because of the damping effect of the medium. Shear fractures are localized in a position where outward displacement from the explosion cavity is maximum. The colored columns of the photograph were

drilled vertically and were uniformly spaced before the blast. The displacement varies in the same manner as the peak pressure. During the interval that the pressure rises in the medium, both the peak pressure and the displacement decreases with distance from the explosive charge. The direction of displacement is outwardly from the explosion cavity during the period of pressure rise. Fig. 4 illustrates the mechanics of deformation following the stage illustrated in Fig. 3. Venting of the gas bubble is followed by a rapid decline of pressure within the explosion cavity and a reversal in the direction of displacement.

FIG. 2.—FRACTURES IN ICE. SHEAR MECHANICS.

RANGES OF SIMILAR BEHAVIOR IN BLASTING

Blast effects are determined by the relation between the energy of the explosion and the mass of material to which the energy is transferred. The chemical composition and weight of the explosive determine the energy of the explosion. The space relations between the charge and the surface determine

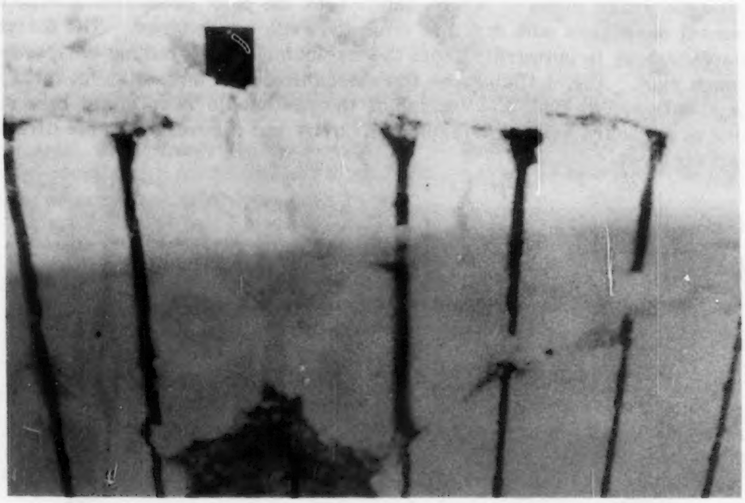

FIG. 3.—SHEAR FRACTURES IN 0.45 DENSITY SNOW.

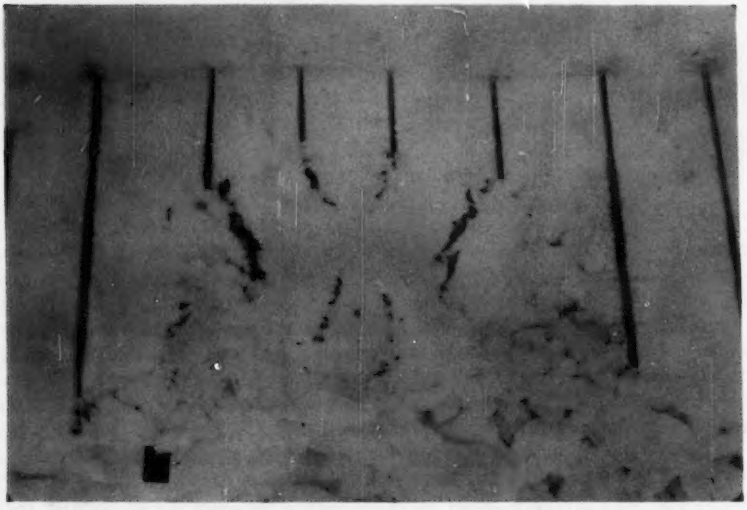

FIG. 4.—DISPLACEMENT OF SNOW DURING THE PERIOD OF PRESSURE DECLINE.

the mass of the material acted upon before failure and the manner in which the energy of the explosion is partitioned either to the material or to the atmosphere. The properties of the explosive and the properties of the material are dependent rather than independent variables, thus both determine the manner in which the material fails.

Research has not yet progressed to the stage where the relations between energy, mass, and time can be stated in absolute units. It is possible, however, to describe blast effects in relation to the weight of the explosive and to the stressed volume within which the event occurs. Such a description leads to recognition of ranges of similar behavior in a wide variety of materials. Similarity in behavior includes geometric similarity relative (a) to the type and degree of fragmentation of the broken material, (b) to the height to which the broken material is thrown, (c) to the noise and airblast that accompanies the explosion, (d) to the volume and shape of the excavation, and (e) to the elastic or plastic behavior of the material. The description leads also to the "energy density concept" in which geometric similarity in materials possessing various physical properties is dependent upon the ratio of the energy partitioned to the material at a given scaled time, and upon the mass of the material within the stressed volume to which the event is referred. The stressed volume is determined both by the depth of the explosive charge and the distance from the charge to the gage at which the event is measured.

Fig. 5 shows the behavior of frozen Keweenaw silt at the lower limit of a range here designated as the "strain-energy range" and the upper limit of a range here designated as the "shock range". The photograph shows (a) a horizontal slab produced as a result of reflection of the shock wave, and (b) vertical radial fractures produced as a result of uplift and stretching of the surface. As the depth of the charge is increased, a point is reached at which the material no longer fails at the surface or is deformed beyond a specified limit. This depth is known as the "critical depth".

The equation (the strain energy equation) that describes the relation between the depth of the charge and the energy of the explosion at the standard of reference to which the energy density within the stressed volume is related, is

$$N = E \sqrt[3]{W} \qquad (1)$$

in which N is the critical depth, in feet, E represents the strain-energy factor (a factor that depends both upon the explosive and the material), and W is the weight of explosive, in pounds.

Rather than to consider the critical depth solely to equal the depth at which failure begins at the surface above the charge, it should be thought of as the depth at which displacement of the surface exceeds a specified limit. By doing so, suitable standards of displacement may be chosen for snow, for soils, for water, and for air. Hence, the strain-energy equation may be applied not only to brittle acting or to plastic-acting solids but also to other materials of the earth's crust.

As the weight of the explosive charge is increased at constant depth, or as the depth of the charge is reduced at constant weight, the surface of the material at the point of reference is deformed beyond the standard. Fig. 6 is a photograph that shows the fracture pattern in frozen glacial till as a limit marking the transition from the "shock range" to the "fragmentation range". Under such conditions a series of slabs of near uniform thickness are formed as the shock wave is reflected from the surface and successively from each of

FIG. 5.—BEHAVIOR OF FROZEN KEWEENAW SILT AT THE LOWER LIMIT OF THE "STRAIN ENERGY RANGE."

FIG. 6.—THE BEHAVIOR OF FROZEN CHURCHILL TILL AT THE TRANSITION FROM THE "SHOCK RANGE" TO THE "FRAGMENTATION RANGE."

the newly formed failure planes parallel to the surface. As may be observed, the slabs are not thrown from the crater as they are formed. Any additional energy available after fracturing the frozen ground causes the action to proceed to the "fragmentation range" in which the slabs are ejected and the particle size is reduced.

The situation illustrated in Fig. 6 is described by the general equation

$$d_c = \Delta E \sqrt[3]{W} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which d_c is the charge depth, in feet, and Δ equals the depth ratio d_c/N.

The depth ratio, Δ, is a dimensionless number that is a ratio of lengths. The numerical value of Δ at critical depth is 1.0. The depth ratio is zero when the center of gravity of the charge is at the surface. The depth ratio is negative when the explosive charge is above the surface. The depth ratio, which is a ratio of lengths may be converted easily to a ratio of volumes, to a ratio of masses, or to a ratio of energy levels—all of which are useful in establishing the fundamental relation between energy, mass, and time for blasts in various materials using various types of explosives.

Thus, it is possible to determine limiting values of the depth ratio at which a transition in behavior of a given material occurs, or to determine the depth ratios at which geometric similarity occurs in different materials with various weights, types and shapes of explosives charges. We thus are provided with a means of determining the energy levels in various materials at which geometric similarity is achieved and at which various new phenomena begin.

Inasmuch as static loading is but a special case of dynamic loading, the energy density concept provides a means of reappraising failure criteria in general, a new avenue of approach to the study of classical theories of failure, and a new avenue of approach towards a more complete understanding of the behavior of materials of the earth's crust.

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NEW TOOLS AND TECHNIQUES FOR DEWATERING[a]

By Byron J. Prugh,[1] M. ASCE

SYNOPSIS

The paper undertakes a description and comparison of the use of present day dewatering tools including conventional wellpoints; high-lift systems with ejectors, submersible or turbine pump units; sand drains; and grout curtain walls. Construction job examples of the use of contemporary dewatering tools are given.

INTRODUCTION

The foreword to a recent handbook[2] on dewatering started out with the phrase, "The science of wellpointing is still an inexact science". The statement may be extended to include all dewatering. But while dewatering is still an inexact science, constant progress has been made in the analysis of soil properties and in procedures for estimating the volume of water to be expected for the dewatering and/or pressure relief of construction excavations. Concurrently with the development of new analytical procedures, the tools used in dewatering and pressure relief systems have been improved and new installation techniques devised. The term tools refers to the equipment and procedures being used to successfully dewater construction excavations.

A dewatering and/or pressure-relief system must be economically evaluated from three specific cost factors:

1. Initial cost of equipment, either rental or purchase.
2. Installation and subsequent removal of the equipment.
3. The operation of the system after installation.

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a Essentially the same as speech presented at May 1959 ASCE Convention in Cleveland, Ohio, entitled "Contemporary Tools for Dewatering."

1 Research Dir., Moretrench Corp., Rockaway, N. J.

2 Moretrench Handbook, April 1958, (Private printing and distribution).

Ironically enough, the first item covering the rental or purchase of the equipment (and transportation) is usually the smallest of the three cost items, yet the equipment cost is often used as a comparative basis in evaluating dewatering systems rather than the overall costs. The latter, which include the installation-operation of the system, would provide a more realistic basis. A contractor often finds that while the rental or purchase cost of equipment is small, the total cost to install that particular system and to operate it for the life of the job far exceeds his original estimate. His purchasing agent has scored a "cost of equipment" reduction, overlooking the engineering and operational factors. On an equal volume basis, several small gasoline pumps will rent for less than one large diesel pump. This apparent saving is reversed when fuel, maintenance, and operating labor costs are considered.

DEWATERING EQUIPMENT COVERS CONSIDERABLE RANGE

Dewatering equipment falls into several broad categories. A multitude of the smaller shallow jobs such as manholes, catch basins, small pipelines, tank pits, etc., or rock and semi-impervious excavations, are done by "open pumping", using either diaphragm or self-priming centrifugal type pumps with the proper utilization of auxiliary devices such as underdrains, filter blankets and sumps. Most foundation engineers discourage open pumping in the vicinity of bearing foundations due to the loosening of the soil by upward water movement. Unless drainage ditches surround the area with deep collecting sumps, the loosening of soil caused by the upward water flow may cause settlement of the completed structure.

The second major dewatering system is the so-called "conventional wellpoint system". Based on the total job dewatering costs, wellpoint systems are generally cheaper than any other method. This holds true even on deep excavations where multi-stage wellpoint installations are used. Each stage of a conventional wellpoint system consists of the wellpoints, usually 1-1/2 in. or 2 in. pipe size, collecting main or "header pipe", discharge piping and wellpoint pumps which consist of centrifugal pumps continuously primed with vacuum pumps.

With proper design, wellpoint pumps may be centrally located. Perhaps the chief advantage of the wellpoint system is its versatility. Soil conditions may not be fully revealed in the original borings and when unexpected dewatering conditions develop, the wellpoint system can be readily adapted to it. Wellpoints can be added in weak locations, additional pump capacity can be supplied and wellpoint screens can be raised or lowered with a minimum of difficulty. Header and discharge pipe can be jobs which are relocated to accommodate construction activities.

Wellpoint innovations of the last decade are now standard in wellpoint design such as: the interconnecting and hydraulic design of several wellpoint stages, at different elevations, to allow one large centralized pump location; the installation of the wellpoint screens in pervious rock formations such as disintegrated mica schist or sandy limestone, by rotary drills or holepunchers; and the use of wellpoints in the consolidation of compressible fine grained soils, or the stabilization of fine grained soils.[3]

Wellpoints may be of any overall length, especially in pressure relief systems where the screen location is governed by the location of the Aquifer. Up

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3 "Stabilization of an Ore Pile by Drainage," by K. Terzaghi and R. B. Peck, ASCE Proceedings Paper 1144, Vol. 83, SM 1, January 1957.

to 70 ft or 80 ft overall wellpoints have been installed using the one third or two point methods of lifting into a vertical position. One of the fundamental limitations of a conventional wellpoint system is the available suction lift per stage which falls between 15 ft and 25 ft, depending upon the design. Therefore, certain types of construction, usually[4] deep excavations with limited physical dimensions (such as tunnels, small deep pits, deep city buildings, etc.) sometimes require the use of the third major dewatering system, temporarily called by the writer, "high lift systems". A "high lift system" employs a pump device in the bottom of a large diameter well screen which removes the suction lift limitation.

All of the deep wellpoint or high lift methods may have a vacuum applied to the casing and through the well screen to the soil. Vacuum application greatly increases the ground water yield from the soil to the well but it simultaneously reduces the capacity of the pumping devices by reducing the available net positive suction head. This vacuum application distinguishes the high lift system from the conventional type of deep well installation which depends on gravity drawdown alone. Pumping equipment for high lift systems is normally of a design intended for water supply installations where the well yield exceeds the pump capacity. On the other hand, in dewatering, the pump capacity must always exceed the well yield and there results a "water-starved" condition which can damage conventional water supply equipment unless precuations are taken.

High lift systems are also applicable to soil consolidation over large areas where a large drawdown and/or partial vacuum application to the underside of a compressible layer is desired. Both types of installations are usually governed by physical rather than economical considerations, and considerable technical knowledge is necessary. Each job must be individually designed. The design of a high lift system should be made by a competent engineer or firm to obtain the maximum efficiency at the lowest cost. Three of the major high lift pump devices are:

Water Ejectors or Eductors.—This is a relatively low cost unit allowing the high lift screens to be spaced closer together at reasonable cost. Individual ejectors have a range of from 5 gpm to 60 gpm. They require the use of two parallel headers, one as a pressure supply, the second as a collecting main. The collecting main discharges through a large vented tank which serves to store water for priming the system. Of great importance is the sizing of the correct ejector to the yield of the wellpoint screen. The motive force is water under pressure with a supply yield ratio of 1:1 or 1.2:1, which actually more than doubles the normal horsepower required to pump a certain gallonage of water from the ground. Even with the optimum sizing of ejectors, the efficiency is only approximately one-third of a centrifugal pump.

For any type of economical operation, ejector nozzles and throats must be changed after the initial pumpdown to obtain reasonable efficiency. For example, as wellpoint yield drops off after prolonged pumping, the ejector internals should be changed to reduce the power input proportionally. Change of internals should also be made if individual pumping tests reveal incorrect sizing of ejector for the ground water yield to the well screen.

Because of power considerations, the maximum practical capacity of an ejector operated high lift system is 1,500 gpm to 2,000 gpm. This usually limits

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4 Liberal use of word "usually" is necessary to avoid arguments due to exceptions in all phases of dewatering.

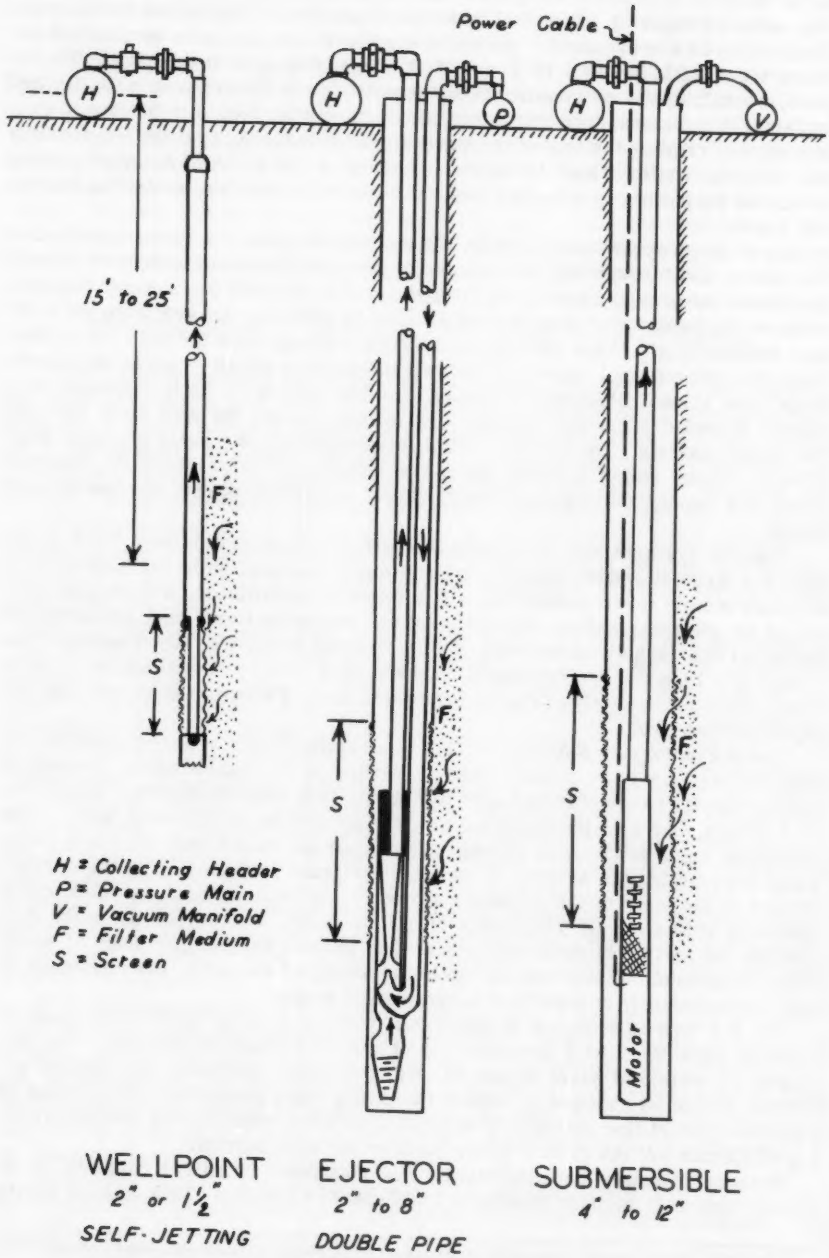

Power Cable
H
H
P
H
V
15' to 25'
F
S
F
S
F
S
Motor
H = Collecting Header
P = Pressure Main
V = Vacuum Manifold
F = Filter Medium
S = Screen
WELLPOINT
2" or 1½"
SELF-JETTING
EJECTOR
2" to 8"
DOUBLE PIPE
SUBMERSIBLE
4" to 12"

FIG. 1

application to soils finer than medium sands. Engine or electric power may be used on centrally located pumps. Two-pipe ejector units have two pipes inside the casing. Single-pipe ejectors have one pipe inside the casing as return and utilize the annular space between casing and return pipe for the pressure supply. Single pipe ejectors are made for 2 in. through 6 in. diameter well casings and have higher capacity in any size than two pipe units. Fig. 1 illustrates schematically the differences between conventional wellpoints and high lift wellpoints.

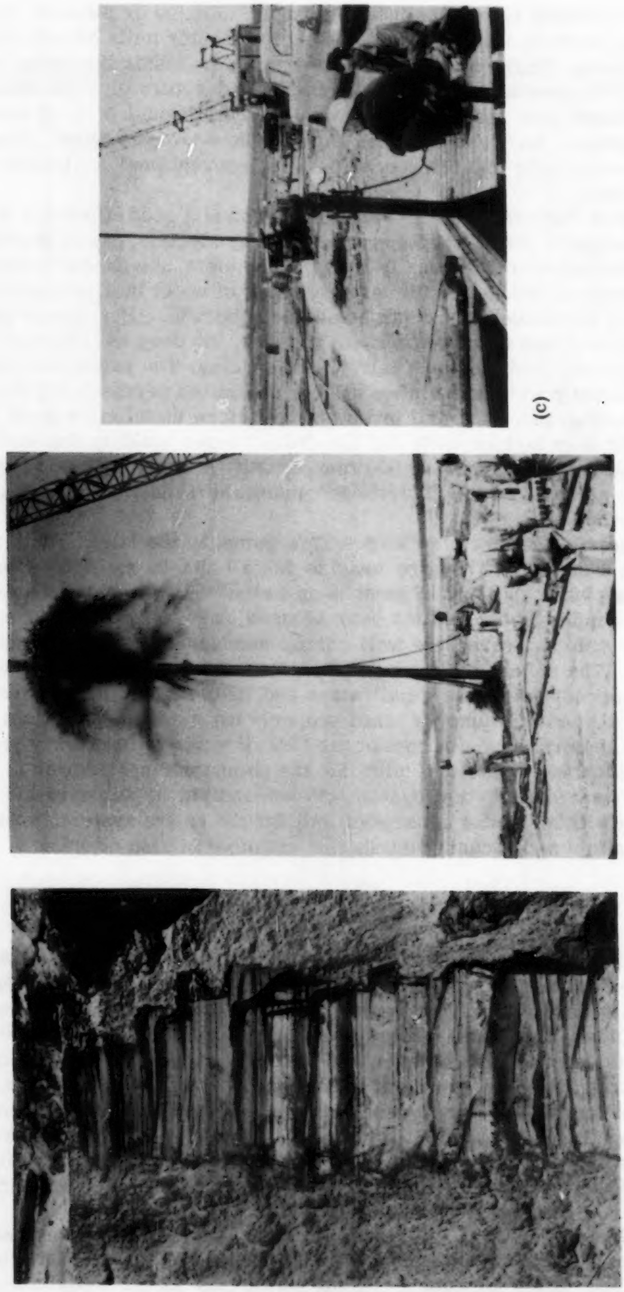
Deep-Well Turbines.—Deep-well turbines have a good efficiency if sized to the deep wellpoint yield. They may be driven by electric, gas or diesel power, but individual power units, usually with standby units, are needed for each well. They are especially feasible for large volumes of water in a very pervious soil extending to a considerable depth below the subgrade. Because of the initial large equipment and installation costs per unit, the deep well turbine pumping device is usually installed on a fairly wide spacing. The subsequent drawdown curves obtained preclude use when a layer of limited permeability is relatively close to subgrade. Careful investigation before installation must be made to correctly size turbine units for the ground water yield to the well screen. Usual range is from 50 gpm to 2,000 gpm per unit. A water collection system is necessary, as is a vacuum distribution manifold if individual vacuum pumps are not used on each well.

The Electric Submersible Pump.—This pump is the latest addition to the dewatering industry. They are easy to install and to remove as the casing need not be plumb like that of most deep wells. In fine-grained soils where smaller diameter submersibles may be used, the placing of the necessary filter sand column around the well casing and screen is a very economical operation. The submersible pump is in some cases cheaper than a deep well turbine pump especially for small sizes and deep setting. They are easy to install, have good efficiency if sized properly but must be run with a shut-off device lest they run dry. A continuous flow of water is necessary to cool the electric motor and to cool and lubricate the pump bearings. Power is supplied by an electrical distribution system to each individual submersible high lift well casing with a standby generating unit for the entire system. A water collection manifold and vacuum distribution manifold is also necessary.

TYPICAL SUBMERSIBLE HIGH LIFT SYSTEM

Fig. 2(a) illustrates the varved fine sand, silt and clay layers in the bank of an 80 ft open cut that had started mov g into a large excavation job located in Canada. The last 10 ft of the varved material was predominantly layers of relatively clean, fine sand. Below was a stable glacial till or "boulder clay" with a few isolated pockets of fine sand. Removal of the free water in the more pervious strata and the lowering of water content in less pervious strata would tend to stabilize the soil and prevent further movement. This was accomplished by the installation of a ten-unit submersible pump high lift system, with individual units pumping from 5 gpm to 50 gpm. The submersible high lift system lowered the ground water from the original level of 20 ft below ground surface to 60 ft over a horizontal distance of 770 ft.

Fig. 2(b) shows an 80 ft long, 12 in. diameter casing being jetted and driven into position by a 100 ft holepuncher. A high lift well screen with riser pipe and the correct filter medium is placed inside this 12-in. installation casing



(a) (b) (c)

FIG. 2

which is then withdrawn leaving the screen, riser and filter correctly positioned in the ground. Piezometers and approximately forty sand drains were installed in a similar manner.

Fig. 2(c) shows preliminary testing of the submersible pump unit after being placed in the well casing, but before connecting to the collection system and vacuum distribution manifold. Upon operation of the system, the mass soil movement ceased in the area of the high lift system.

NEW TECHNIQUES

Besides the improvements in dewatering equipment and installation techniques previously mentioned, two other techniques are used as supplemental aids to dewater construction excavations.

Sand Drains.—The first of these is sand drains already used extensively for soil consolidation. Sand drains are also installed in stratified or laminated soils that contain alternate layers of pervious and relatively impervious soil. The sand drains conduct water from the higher permeable layers to the lower permeable layers where the dewatering screens are located. This is of great economic importance in reducing overall dewatering costs. For example, a pipeline trench in sand that has a horizontal semi-impervious layer above subgrade, may be dewatered with equipment on one side of the trench if sand drains are placed on the other. In addition, sand drains are used to stabilize the slopes (or banks) of construction jobs, to decrease excavation, slope area, material handling distance, to increase safety and working area and to better working conditions. An example combining both applications, is an excavation made in a swamp near Lake Charles, La. Soil conditions at the time of construction were:

6 ft of dredged fine to medium sand fill;
20 ft of compressible vegetable and organic silty clay with peat, roots, etc.;
4 ft of medium compact silty clay or fine sand with clay lenses;
20 ft of dense, very uniform fine sand.

Subgrade was 14.5 ft from the surface with 8.5 ft of water to be removed. The original foundation report recommended not more than the 6 ft of fill to avoid shear action in the 20 ft of compressible material. A settlement of 2-1/4 ft was expected in the compressible layer in 2 yr due to the 6 ft of fill. Structure was to be supported on piles driven into the dense fine sand. Structure bearing design was no problem, but the construction excavation was.

Dewatering, bank stabilization and reduction of excess pore pressures in the compressible layer was accomplished by surrounding the excavation area with 38 ft long wellpoints that extended from water level into the dense, very uniform fine sand layer (aquifer). Header pipe was located at water level with a 2 horizontal to 1 vertical slope to the outside structure line. A 39 ft long casing with teeth (bog cutter), was jetted in on 5 ft centers to create a 10 in. diameter hole, in each of which was placed a 2 in. wellpoint surrounded by a uniform medium sand as a filter medium. Three rows of sand drains were also installed, one 15 ft inside the header, the others 15 ft and 30 ft outside the header with the sand drains staggered on 15 ft centers in each row. (Fig. 3).

Use of a jetted casing for the installation of the sand drains rather than a conventional driven casing with end closed with flap release, eliminates the "smear" or remolded zone adjacent to the sand drain. This allows horizontal

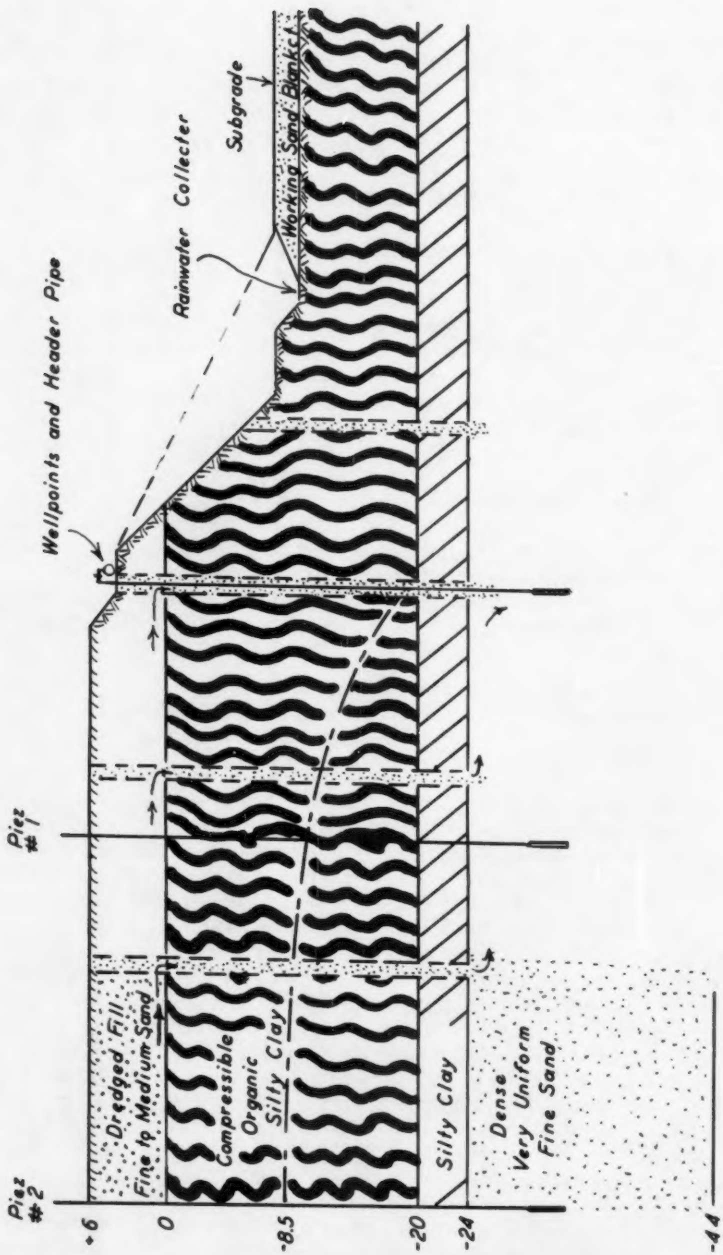


FIG. 3.—SAND DRAINS AND WELLPOINTS

permeability or flow to approach that of the normal ground rather than undergo reduction, speeds up the time necessary for consolidation and reduces the buildup of excessive pore pressures. Other factors being equal, this will reduce the time to one half or less.[5] Advantage may also be taken in reducing the sand drain diameter and increasing the spacing.

The filter medium used for filling the sand drains did not follow the usual arbitrary pattern of a coarse sand to fine gravel such as has been specified on most previous sand drain work, but was instead a very uniform fine sand, D_{50} = 0.23 mm, U_c = 1.3 with an estimated inplace permeability of 300 mu per second. The filter medium was designed on the basis of being at least twenty-five times as pervious as the fine grained compressible soil; of being fine enough to prevent intrusion of the surrounding soil; and as uniform as possible to reduce segregation to a minimum when placed in water. The locally available fine sand met these requirements and was extremely economical. The contractor was able to scoop this up locally, eliminating the necessity of purchase and transportation.

Fig. 4(a) illustrates the stabilized bank with dragline excavating. Wellpoint pumps are visable to the rear of dragline.

Fig. 4(b) is a general view showing wellpoints and pile driving rig. Both photographs were taken after a heavy rain but both show the bank slopes that were steeper than the one on two originally contemplated. After the excavation was completed, a hurricane passed over the job completely filling the excavation with water. When pumped out, the banks were still stable. Performance of the economical fine sand as a filter in the sand drains was excellent. The high organic content of the compressible layer prevented comparison of the grain size ratio with pervious published results[6] of hydraulic gradient losses in vertical filters. Maximum consolidation was obtained near the wellpoint header.

Grout Curtain Wall.—Under certain conditions it may be advisable to alter the permeability of the existing soil by the use of grout. One such condition would be a very permeable rock or gravel formation where the soil permeability is over 2000 mu per sec. If the hydraulic head differential is large, the volume of water that must be pumped to dewater the job may be economically impractical to handle. An inexpensive, semi-plastic, cement-clay grout curtain wall will reduce the flow to such a degree that the small seepage remaining may be handled by normal dewatering equipment. The total cost of dewatering being far less than either a complete grout cut-off wall or pumping. For relatively shallow excavations, a trench excavated under water and, as excavation proceeds, continuously refilled with an impervious clay or clay-cement slurry may be feasible. Deeper excavations require the use of grout pipes for the curtain wall placement. The second case is of a deep excavation of limited extent where the grout used will depend on the soil properties of the material to be grouted. Grouting effectiveness in this case may range from partial to total although the intent is usually for a total water cut-off. If correctly installed, a grout curtain wall or grouting needs no maintenance.

An example of the first case was a power plant constructed on the banks of the Ohio River where a deep well pumping test in the 50 ft of sand and gravel over rock, gave permeability values of from 5,000 to 6,000 mu per sec. Sandy

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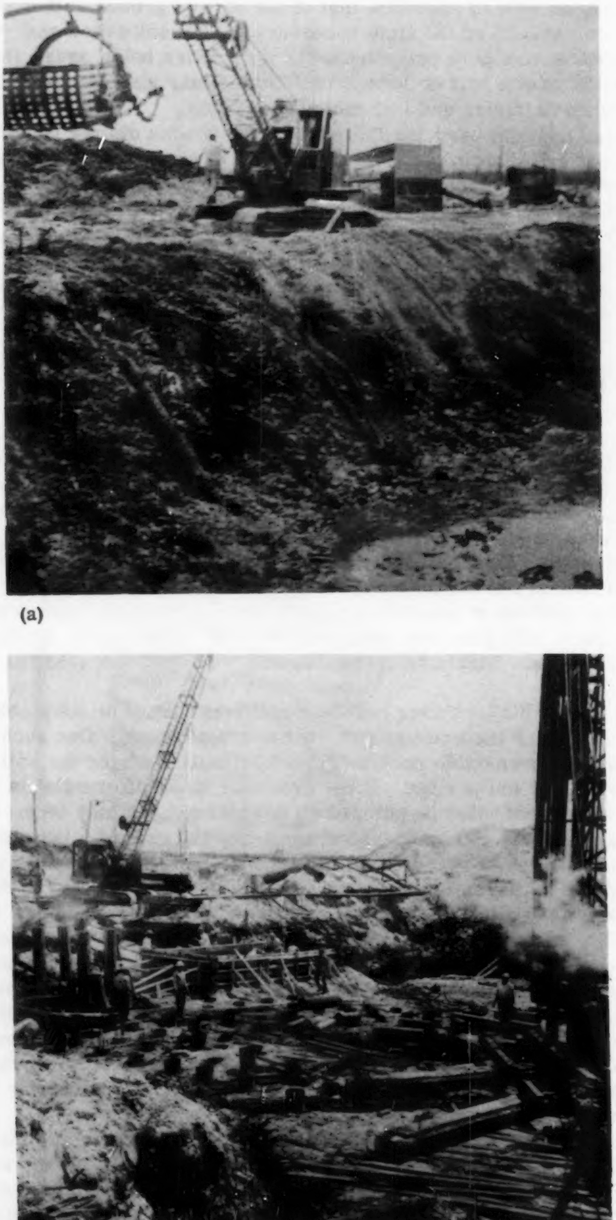
5 "A Review of the Theories of Sand Drains," by F. E. Richart, Jr., Prof. Civ. Eng., Univ. of Florida, ASCE Proceedings Paper 1301, Vol. 83, SM 3, July 1957, page 11-18.

6 "Experiments on Uniformly Graded Filters," by Heinz Zweck and R. D. Daridenkoff, Proceedings 4th International Soil Mechanics Conference, London, 1957, Vol. II, page 410.



(a)

(b)

FIG. 4

clay (loam) (20-ft deep) extended from the sand and gravel to the ground surface. Dewatering volume was estimated to be from 15,000 gpm at pool to 50,000 gpm at flood stage if an earth cofferdam only was used. Steel sheeting was suggested but was not readily available.

A curtain wall of 1,700 ft was grouted in the shape of a U". While the 70 ft grout pipes were installed from the original ground level, only the entire thickness of the sand and gravel layer overlaying the rock, varying from 20 ft to 50 ft in thickness, was grouted for a vertical surface area of approximately 50,000 sq ft. Cost was considerably less than a single row of steel sheet piling. Despite some installational errors, including one hole in the grout curtain wall you could "drive a truck through", the volume of water pumped was reduced to approximately 30% of the minimum expected.

There were roughly, 800,000 gal of grout pumped in through 10,000 lineal ft of grout pipe or some 16 gal per sq ft of curtain wall. This is equivalent to a wall 5 ft thick based on a 40% porosity. A conservative average of 30,000 gal was pumped per shift which required the use of approximately 4 tons per hr of dry material. Grout was mixed in batches of 4,000 gal in two alternately used flocculation tanks. Pumping was at the rate of from 75 gpm to 200 gpm.

An admixture of certain salts and dyes was added to the cement-clay grout which imparted color so that mixtures could be readily identified, strength increased and the flocculation and set times could be varied. The cement-clay grout when set, while semi-plastic, has no thixothropic properties as exhibited by pure bentonite grouts. After setting it will not move under hydrostatic pressure.

Fig. 5(a) shows typical stratified soil ranging from coarse sand to well graded sand and gravel to uniform medium gravel.

Fig. 5(b) shows the top of the hole puncher used to install grout pipes, grout pipes and the grout batch plant in the background.

Fig. 6(a) shows the grout batch plant with Athey wagons bringing material through typical Ohio Valley mud. Roof had been blown off the previous day in a rain and wind storm. Note materials stacked on tank for next batch.

Fig. 6(b) shows crane pulling grout pipes from first row. Spilled grout shows white on ground. Second row of grout pipes in the foreground was not used. Crane is located between screen house and river on top of dyke. Approximately 2 yr after the initial installation of the grout curtain wall, it was able to be visually inspected to determine its effectiveness as a dewatering aid. Observation was made when the dyke was partially removed, eliminating any possibility of a silt blanket over the river bottom.

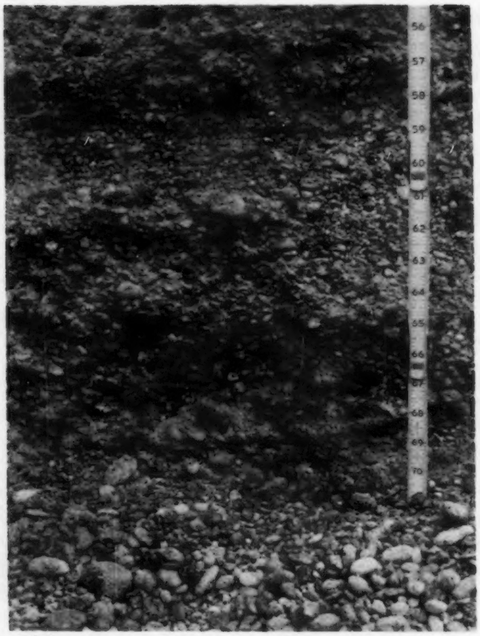
Fig. 7(a) shows a test pit dug by a floating rig on the river 12 ft away. Major pumping had stopped in the area and water level was being held 10 ft below river some 200 ft from test pit by pumping equipment yet the test pit is dry 8 ft at this spot. Grout penetrated 3 ft into coarse sand, 6 ft to 10 ft into openwork gravel.

Fig. 7(b) shows a cross section through the test pit with observed conditions.

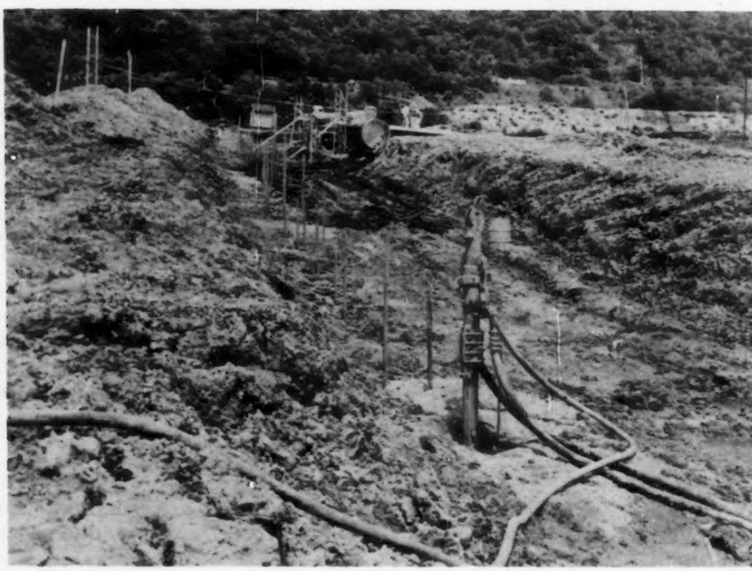
CONCLUSIONS

1. Tools and methods to be considered when designing a construction dewatering system primarily depend on the soil properties at the site and the physical limitations of the job.

2. Overall dewatering costs should be considered rather than one item, such as rental of dewatering equipment. By altering the dewatering method



(a)



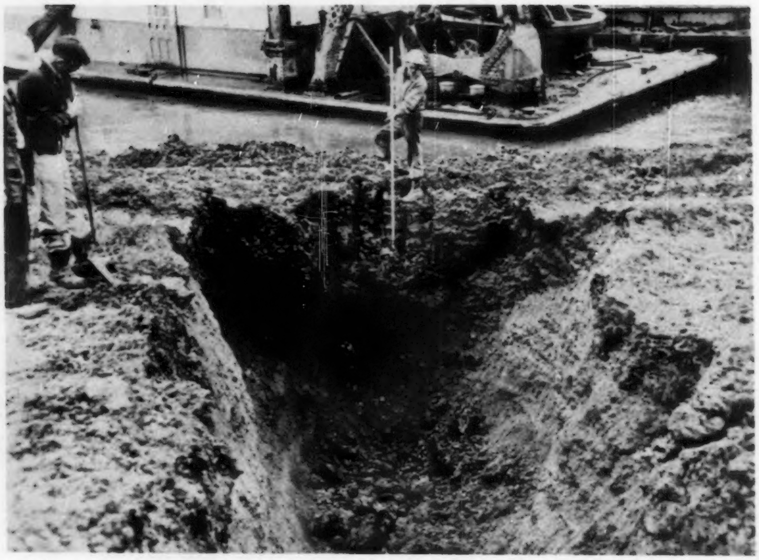
(b)

FIG. 5

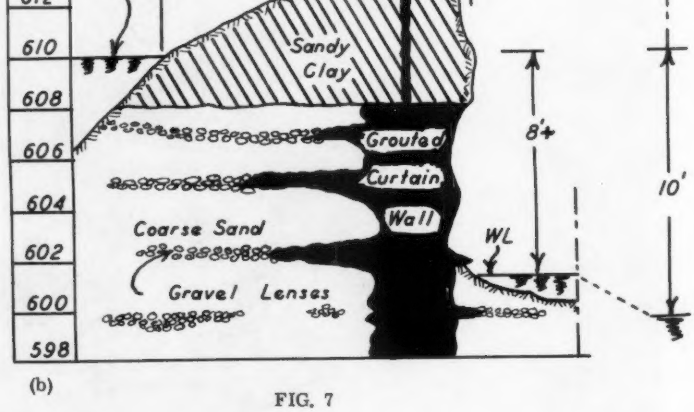
(a)

(b)

FIG. 6



(a)


ELEV.
614
612
610
608
606
604
602
600
598
12'
200'
Pool
Sandy Clay
Grouted
Curtain
Wall
8'+
10'
WL
Coarse Sand
Gravel Lenses

(b)

FIG. 7

and tools, overall savings may be made on the job due to reduction in labor, excavation or materials.

3. All tools and methods should be considered in dewatering design and may include any of, or combinations of, (a) open pumping from sumps, (b) conventional wellpoints, (c) conventional deep well type pumping, (d) high lift systems with either ejector, submersible or turbine pumping units, (e) sand drains, and (f) curtain wall cut-offs.

4. All dewatering jobs should be designed by qualified personnel to fit the individual conditions of that particular job and to obtain maximum results at a minimum cost. Where total dewatering costs are particularly high, such as on very large or deep excavations, extensive engineering analysis in advance is warranted to determine the most suitable and economic method.

Journal of the
CONSTRUCTION DIVISION
Proceedings of the American Society of Civil Engineers

FIELD OFFICE BOOKSHELF

Progress Report of the Committee on Publications of the Construction Division

Every other year Contractors and Engineers publishes a revised bibliography listing of books recommended and suggested for a Field Office Bookshelf compiled by M. D. Morris, F. ASCE. This year, through the courtesy of the magazine and bibliographer, the lists were reviewed and brought up to date by the Publications Committee of the Construction Division and are presented herewith.

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 1, February, 1960.

RECOMMENDED VOLUMES

NAME	PUBLISHER	PRICE
1. GENERAL ENGINEERING HANDBOOK C. E. O'Rourke, 1940	McGraw-Hill 330 W. 42nd St. NYC 36	$14.00
2. AMERICAN CIVIL ENGINEERING PRACTICE Robert W. Abbett 1956-1957	J. Wiley & Sons 440 4th Ave. NYC	3 vols. $15, $15, $25
3. BLASTERS HANDBOOK 1954	E.I. DuPont de Nemours Wilmington, Del.	$1.80
4. PILE FOUNDATIONS R. D. Chellis, 1951	McGraw-Hill	$14.00
5. MOVING THE EARTH Nicholas, 1955	D. Van Nostrand 250 4th Ave. NYC 3	$15.00
6. CONSTRUCTION COST CONTROL Maxton, Babb, Leitch, 1955	ASCE 33 W. 39th St. NYC	$4.00 Members $5.00 Non-Mem
7. SUPERVISOR'S SAFETY MANUAL	National Safety Council 425 N. Mich. Ave. Chicago 11, Ill.	
8. CONSTRUCTION PLANNING, EQUIPMENT & METHODS R. L. Peurifoy, 1956	McGraw-Hill	$8.50
9. UNDERPINNING Prentis & White, 1950	Columbia Univ. Press New York 27, N.Y.	$10.00
10. STEEL CONSTRUCTION MANUALS 1956	AISC 101 Park Ave NYC 17	$3.00 (3.50-thumb index)
11. CONCRETE MANUAL	Bureau of Reclamation Supt. of Documents U.S. Govt. Printing Office Washington 25, D.C.	$2.50
12. ASPHALT HANDBOOK 1951	The Asphalt Institute College Park, Maryland	
13. CIVIL ENGINEERING HANDBOOK Urquhart, 1959	McGraw-Hill	$17.50
14. MECHANICAL ENGINEERS HANDBOOK Marks, 1958	McGraw-Hill	$23.50
15. MINING ENGINEER'S HANDBOOK Peele (2 vols.), 1941	J. Wiley & Sons	$18.75
16. ST'D. HANDBOOK FOR ELECTRICAL ENGINEERS, Knowlton, 1957	McGraw-Hill	$21.00
17. CHEMICAL ENGINEER'S HANDBOOK J. Perry, 1950	McGraw-Hill	$21.00

	NAME	PUBLISHER	PRICE
18.	RAILWAY TRACT & STRUCTURE CYCLOPEDIA, 1955	Simmons Broadman 60 Church St. NYC	$6.00
19.	PHYSICS Houseman & Slack, 1957	D. Van Nostrand	$8.00
20.	PIPING HANDBOOK S. Crocker, 1945	McGraw-Hill	$15.50
21.	MANUAL OF ENGINEERING DRAWING T. E. French, 1953	McGraw-Hill	$9.25
22.	LEGAL ASPECTS OF CONSTRUCTION Walter C. Sadler, 1959	McGraw-Hill	$8.50
23.	ENGR'S. DICTIONARY (Spanish-Eng. & V.V.) (or equiv.) L. A. Robb, 1949	J. Wiley & Son	$12.50
24.	MATERIALS HANDBOOK (8th EDITION) G. S. Brady, 1956	McGraw-Hill	$13.50
25.	METALS HANDBOOK - 1948 American Soc. for Metals, 1955 (with 1st Supplement)	A.S.M. 730 Euclid Ave. Cleveland, Ohio	$15.00 Members $21.00 Non-Mem
26.	SOCIETY OF AUTOMOTIVE ENGRS. HANDBOOK, 1959	S.A.E. 33 W. 39th St. NYC 18	$10.00 Members $20.00 Non-Mem
27.	HYDRAULIC HANDBOOK 1955	Fairbanks Morse & Co. Chicago 5, Illinois	$3.50
28.	EQUIPMENT RENTAL RATES Issued yearly	Associated Equipment Distributors, 30 E. Cedar St. Chicago, Ill.	$5.00
29.	NATIONAL DESIGN SPECIFICATIONS for STRESS-GRADE LUMBER AND ITS FASTENINGS	National Lumber Mfr's Assoc. Washington, D.C.	
30.	SOIL TESTS FOR - CONSTRUCTION G. Bertram (#107)	American Road Builders Assoc. World Center Bldg. Washington, D.C.	$1.00

OPTIONAL ADDITIONAL VOLUMES

1.	PLANT ENGINEERING HANDBOOK W. Stanier, 1959	McGraw-Hill	$23.50
2.	PRODUCTION HANDBOOK Alford, Bangs, 1944	Ronald Press 15 E. 26th St. NYC	$12.00
3.	HANDBOOK OF REFRIGERATING ENGRS. Woolrich-Bartlet, 1948	D. Van Nostrand	$12.75
4.	PRESTRESSED CONCRETE G. Magnel, 1954	McGraw-Hill	$9.75

NAME	PUBLISHER	PRICE
5. HEATING, VENTILATING AIR CONDITIONING GUIDE, 1959	Am. Soc. Heat. & Vent. Engr. 62 Worth St. NYC 13	$9.00
6. ILLUMINATING ENGINEERING SOCIETY LIGHTING HANDBOOK, 1959	I.E.S. 1860 Broadway, NYC 23	$10.00 Non-Mem $7.50 Members
7. COMMUNICATION ELECTRONICS Fender & McIlwain, 1950	J. Wiley & Sons	$10.00
8. ELECTRICAL TRANSMISSION & DISTRIBUTION REFERENCE BOOK 1950	Westinghouse Elec. Co. 40 Wall Street N.Y.C., N.Y.	$6.00
9. MAKING, SHAPING & TREATING OF STEEL, 1951	U.S. Steel Co. Pittsburgh, Pa.	$7.50
10. NATIONAL ELECTRICAL CODE, 1959	Nat'l Fire Protection Assoc. 60 Batterymarch Boston 10	$1.00
11. NATIONAL FIRE CODES, 1959	Nat'l Fire Protection Assoc.	$7.00
12. CONVERSION FACTORS & TABLES Zimmerman & Lavine, 1955	Industrial Research Service Dover, N.H.	$5.00
13. AMERICAN PIPE MANUAL 1951	American Cast Iron Pipe Co. 117 Liberty St. NYC	N/C
14. COMPRESSED AIR & GAS INSTITUTE HANDBOOK, 1954	McGraw-Hill	$3.00
15. DIESEL ENGINEERING HANDBOOK	Diesel Publications, Inc. 192 Lexington Ave. NYC	$8.50
16. COMBUSTION ENGINEERING 1947	Combustion Engineering Co., Inc. 200 Madison Ave. NYC 16	$7.50
17. BUILDING CONSTRUCTION HANDBOOK Frederick C. Merritt, 1956	McGraw-Hill	$15.00

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Journal of the
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POLYVINYL ACETATE AND PORTLAND CEMENT MORTARS

By Robert T. Howe,[1] M. ASCE

SYNOPSIS

Polyvinyl acetate, a synthetic resin widely used as a base for paints and adhesives, offers some promise as an admixture for portland cement mortars. This paper describes polyvinyl acetate, and outlines research on its possible use in a concrete floor surfacing material. Full-scale applications made several years ago are described and their present conditions evaluated.

INTRODUCTION

In 1950, at the suggestion of a large manufacturer of chemicals,[2] a Cincinnati manufacturer of cement products[3] sponsored a research project on the effects of polyvinyl acetate on portland cement mortars. At that time, the sponsor was planning to build an all-concrete house, and had hopes that the use of this plastic might produce "resilience[4]" in his concrete floors.

The writer was engaged to carry out this study through the University of Cincinnati Research Foundation from December, 1950 through March, 1953.

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 1, February, 1960.

[1] Assoc. Prof. of Civ. Engrg., Univ. of Cincinnati, Cincinnati, Ohio.

[2] E.I. duPont de Nemours & Co., Inc.

[3] Arthur C. Avril, Pres., Sakrete, Inc.

[4] When "resilient" is used in connection with flooring, it would seem to mean the ability to absorb energy without permanent deformation, or essentially inelastic behavior in contrast to the technical meaning of the word.

During this period, many possible mixes and uses were investigated, and seven major applications of these mixes were made, including the surfacing of the entire floor of the sponsor's home. This paper (a) describes polyvinyl acetate, and what is known of its actions in cement mortars, (b) discusses procedures of use which have been found to be desirable, and those which are undesirable, and (c) reports on the current status of seven major applications of such mortar.

POLYVINYL ACETATE

Polyvinyl acetate is a synthetic, thermoplastic resin, which is widely used in paints and adhesives. In pure form, it takes the shape of small, clear, glass-like beads which are not soluble in water, and which are comparatively inert to many other chemicals.

For many commercial purposes this plastic is emulsified during the manufacturing process. This operation gives rise to a rather viscous, milk-white fluid, which smells somewhat like vinegar and has a pH ranging from 4 to 6. When this emulsion is spread into a thin film on a clean, hard surface, the milkiness disappears as the water evaporates, and a very tough, transparent film is left tightly adhering to the surface. This film is quite resistant to attack by petroleum products, glycerine, turpentine, and other chemicals, but is soluble in many alcohols. If this film is merely touched with a drop of water, it will almost instantly turn milky, and, in a very short time, will lose its adhesive properties. When a large drop is placed on a hard clean surface, the surface of the drop will tend to polymerize and form a film over the interior of the drop, thereby preventing the interior from solidifying.

Polyvinyl acetate emulsions are now made by several manufacturers in a variety of forms. "Elvacet" 81-900[5] was used almost exclusively in the research project, and in the balance of this paper Elvacet will mean the 81-900 form of polyvinyl acetate emulsion.

INTERACTIONS OF POLYVINYL ACETATE AND PORTLAND CEMENT

Several important phenomena arise when polyvinyl acetate and portland cement are combined. Many references were consulted through 1952 in an effort to explain these phenomena, but the information found could only be used as a basis for some deductions, which seem to be valid.

Since portland cement and Elvacet are both binding agents, it may appear to be unnecessary to use them together, but these two materials do complement each other within certain limits. It is well known that cement mortars are stronger when the cement particles are more completely dispersed. A microscopic examination of a "thin section" of hardened mortar containing the plastic seemed to indicate that the plastic becomes the "continuous phase" in the matrix, and that the cement particles do tend to hydrolyze individually. Conversely, as noted above, the emulsion cannot polymerize completely unless the water can escape from the center of the mass, and the cement is available to take up some of this water through the process of "setting".

As noted above, although polyvinyl acetate is essentially insoluble in water, the application of only a small amount of water to a hardened film, formed from

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5 "Elvacet" is a brand name of duPont for emulsified polyvinyl acetate. 81-900 indicates the degree of polymerization.

the emulsion, will quickly soften this film. If an alkaline substance is added to this drop of water, the polymerized acetate will tend to hydrolyze into polyvinyl alcohol, which is also a good adhesive but quite soluble in water. At the same time, a salt is formed from the mineral portion of the alkali.

The last characteristic of the plastic which must be noted is its need for light and/or heat to catalyze its polymerization.

The above characteristics of the plastic mean that mortars containing polyvinyl acetate cannot be used in the same ways as plain mortar. Of greatest significance is the fact that such mortars cannot be moist-cured, and should not be covered while curing. This means that the material can be spread over a floor, for example, and left to cure without further attention. It also means that applications of the mortar cannot be too thick or the lower portions will not set up completely.

Effects of Interactions on Hardened Mortar.—As stated above, this research project was initiated in the hope of finding a resilient floor surfacing material. Since no standard test of this type of resilience could be found, it was decided that tensile strength would probably come closest to indicating the desired characteristics. Standard mortar tension briquettes were, therefore, the principal form of test specimen made, although many 2 in. cube and 2 in. diameter by 4 in. cylinders were also used. Many combinations of sand, cement, water, and Elvacet were tried in the course of the investigation, but whenever the term "mortar" is used in the balance of this section, it will refer to the combination of ingredients which will later be defined as the "standard mix".

Tension briquettes removed from the molds 24 hr after casting, and allowed to stand on edge on a window ledge for 6 additional days, would almost always break at a minimum of 700 psi. Similar samples stored for the 6 days in a closed drawer would break at about 500 psi, while samples cured in water for any length of time always broke at about 150 psi. The compressive strengths of the cubes and cylinders were seldom greater than would be expected of ordinary mortars, being about 2500 psi at 14 days. In most cases, the material at the center of a newly broken compression specimen would be somewhat moist, indicating that the surface had hardened before all of the excess moisture has had a chance to escape. All of these observations, plus experience with full-scale installations, indicate that mortars containing polyvinyl acetate should be applied in layers approximately 1/2 in. thick, and allowed to cure in as light, and dry, a place as possible.

When hardened tension briquettes of the standard mix were immersed in water, moisture would penetrate into the material very slowly, but after 24 hrs, or so, a briquette would be moist throughout, and would have a tensile strength of about 150 psi. Samples which were cured in the air for 7 days, immersed in water for 7 days, and re-dried in the air for 7 days, would regain their strength, up to about 500 psi.

Whenever a mortar sample containing Elvacet was immersed in water for more than 1 day, the surface of the sample would become rather soft and could be easily scratched with a knife. At the same time, a white, flaky "scum" would appear on the surface of the water. While the amount of this scum increased with increased time of immersion, its total thickness never exceeded a few 1/100ths of an inch. Limited qualitative analysis of this scum seemed to indicate that it was probably calcium acetate, formed during the hydrolyzation of the polyvinyl acetate in the presence of the water and alkali of the portland cement. Repeated immersions and air-dryings would continue to produce small traces of the "scum", and would cause the tensile strength to vary from 150 psi when wet to 500 psi when dry.

Attempts to Inhibit Hydrolysis of Polyvinyl Acetate.—Much effort was exerted in an effort to find a way to inhibit the reaction of the hardened mortar when submerged in water. "Chemical Abstracts"[6] were searched for the years 1946 through 1952 for ideas, and many procedures and admixtures were tried. Among others, the following ideas were explored: (1) substitution of gypsum for portland cement, (2) substitution of high-aluminous cement for portland cement, (3) the direct application heat during curing, (4) addition of traces of the following chemicals; (a) steratochromic chloride in isopropanol, (b) calcium chloride, (c) chromic oxide, (d) potassium chromate, (e) benzoyl peroxide, and (f) aluminum chloride.

It must be stated that none of the attempts to inhibit the hydrolysis succeeded. Some of the techniques and admixtures produced very undesirable results, and many of them reduced the strength of the mortar. The susceptibility of polyvinyl acetate emulsions to hydrolysis is probably the greatest problem in its possible use in mortars.

STORAGE AND HANDLING OF EMULSION

Elvacet can be purchased in containers varying in size from one-quart bottles to railroad tank cars. While slightly acidic, minor contact with it will not hurt human skin. It can be handled in about the same way as SAE 30 oil.

The manufacturer recommends that the emulsion, as sold, be stored for not more than 4 months before it is used, and that it certainly be stored at temperatures above freezing to prevent the breaking of the emulsion. It is also recommended that diluted forms of the emulsion be used immediately to prevent coalescence of the plastic resins.

An effort was made to stabilize the emulsion against the effects of freezing and storage in diluted form. It was found that the addition of 2% bentonite, by weight of the diluting water, would prevent a mixture of 40% Elvacet, by volume, and 60% water from coalescing through at least 8 cycles of freeze-thaw, and storage of at least one month. When subjected to these conditions, the plastic solids and bentonite particles settled out of suspension fairly quickly, but they could be easily re-dispersed by merely shaking the container. In general, the 7-day tensile strengths of mixes containing the bentonite were about 500 psi, but were sometimes higher, depending upon the conditions of mixing and curing.

LABORATORY MIXES

The "standard mix" will now be described, and the characteristics of a variety of other experimental mixes will be compared with those of the standard mix. Throughout the balance of this paper, it should be kept in mind that the goal of this research project was to achieve, if possible, a blend of plastic and conventional mortar materials which would give satisfactory results under conditions of mixing and application which might reasonably be attained in the field. Dry ingredients were proportioned on the basis of weight, liquid ingredients on the basis of volume, liquid and dry on a basis equivalent to the water-cement ratio, and no effort was made to measure to the ultimate of laboratory precision.

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6 CHEMICAL ABSTRACTS, published be-monthly by the American Chemical Society.

The standard mix contained dry ingredients made up of 30% portland cement and 70% medium-grained sand by weight, and liquid ingredients blended from 40% Elvacet and 60% tap water by volume, with the liquid and dry ingredients combined in the equivalent of 5 gal of liquid per sack of cement. On a dry-weight basis, about 0.11 lb of plastic resin was used per 1.00 lb of cement. This mix had good workability, and very satisfactory strength characteristics, as discussed above. Laboratory samples were mixed in a pan by hand with a mixing spoon, while mortar boxes and hoes were used for the full-scale installations. This mix was somewhat "spongy" under a trowel. It tended to stick when a trowel was flat on the surface, but could be easily smoothed with the edge of a trowel.

Other combinations of sand and cement with Elvacet, ranging from all sand to all cement, were made into tension briquettes and tested. Those with less than 25% cement tended to stretch and were not as strong as the standard mix, while those with more than 35% cement tended to shrink more during the setting and to be about the same strength as the standard mix.

Mixing liquids ranging from all water to all Elvacet were also tried. Specimens made with liquids containing less than 30% Elvacet, by volume, were appreciably weaker than those of the standard mix, while those with successively higher percentages of Elvacet became progressively more sticky and difficult to work, without compensating gains in strength.

Liquid-to-cement ratios ranging from the equivalent of 1 gal per sack to 6 gal per sack were used, and, in general, the tensile strength was found to vary in about the same way as the compressive strength of ordinary mortar. The maximum strength, 800 psi, was obtained at 4 gal per sack, but this mix was very stiff. By the time the liquid-to-cement ratio reached 6 gal per sack, the mix was too soft, and the tensile strength had dropped below 700 psi.

Since almost all mortars tend to shrink somewhat during the setting period, it would be expected that mortar containing polyvinyl acetate might also tend to shrink. While this shrinkage was no greater than that of ordinary mortar, some effort was made to blend gypsum into the mix to eliminate that which did occur. It was found that when gypsum was substituted for a portion of the portland cement, shrinkage was eliminated, but, unfortunately, specimens containing this material tended to swell and crack when immersed in water long enough to become soaked through.

In addition to the materials mentioned thus far, several others, including white-waterproof cement, diatomaceous earth, and agricultural lime, were used in various blends, but no other combination of ingredients gave as consistently strong and workable a mortar as did the standard mix.

LABORATORY EXPERIMENTS

In addition to the tensile and compressive tests, a variety of non-standard experiments was run on the mortar.

To investigate the problem of curing a more massive sample, two #2-1/2-size tin cans were filled with the standard mix, and allowed to stand with the top surface of the mortar exposed to the light and air. Four days after the cans were filled, a knife blade could still be inserted into the material with comparative ease. The metal was then peeled off and the mortar was exposed to the air for three more days. At the end of this period, the surface was quite resistant to abrasion, but, when the blocks were broken, the interiors were found to be rather moist.

Flexural tests were run on 2 in. x 2 in. x 24 in. beams cast from the standard mix, and on 3/4 in. x 2 in. x 24 in. wood strips overlain with 3/8 in. thicknesses of standard mix. The 2 in. x 2 in. x 24 in. beams were tested in three different ways, with the following results: (1) a beam made with a liquid to cement ratio somewhat higher than the standard mix failed under a mid-span load which produced a deflection of almost 0.10 in., and an extreme fiber stress of 740 psi; (2) a load of 150 lb was applied at mid-span (maximum fiber stress of 675 psi) and maintained for 28 days, after which the beam was found to be broken, but the testing machine was in an open location, and the real cause of failure is not known; (3) a beam was alternately loaded with 150 lb at midspan and unloaded through 2,000 cycles, after which the load was increased to 180 lb (maximum fiber stress of 810 psi) and 500 applications made before the beam broke. When the wood strips were coated lightly with diluted Elvacet before the mortar was spread over them, the mortar bonded so tightly to the wood that failure always occurred in the tension side of the wood (the mortar was on the compression side), even when the beams deflected as much as 1 in.

Several other experiments were run to test the bonding ability of the mortar, but only one crude test will be mentioned. The sheet-metal conduits of many dust collecting systems are subject to a great deal of abrasion. With the idea that the standard mix might provide a suitable lining material for such ducts, an 8 in.-long section of 3 in.-diameter galvanized sheet metal pipe was primed with a thin coat of Elvacet and then coated with an 1/8-in.-thick layer of mortar. After the mortar had set, the pipe was squeezed many times by hand, but not a spot of mortar dropped away from the metal.

FULL-SCALE INSTALLATIONS

Seven major applications of mortar containing polyvinyl acetate were made between July, 1951 and July, 1953. The first installation to be described contained some perlite in place of about 20% by volume of the sand, while all of the others were made from the standard mix.

Six of the jobs were done by a small local contractor employing one finisher and from one to three laborers. No attempt was made to control the quality more closely than would be done on an ordinary job. The dry ingredients were premixed in the plant of the sponsor. The emulsion was diluted on the job by measuring out 2 cans of emulsion and 3 cans of tap water. Since the 5-gal-of-liquid-per-sack-of-cement ratio yields very nearly the optimum workability, and since this workability is rather sensitive to the amount of liquid, it was found possible to let the laborer, mixing the material with a hoe in a mortar box, decide how much liquid to add to a given batch.

In July, 1951, the sponsor built a new house of concrete construction, with radiant heating coils in the concrete slab. The base slab was leveled to within about 1/4 in., and given a rough float finish. A mortar approximating the standard mix, but containing some perlite, was used to finish the 3,000 sq ft of floor area. Air temperature during application varied from 75° F to 95° F, and the rooms were quite light, although the floor received very little direct sunlight. Because of the base slab, the mortar varied in thickness from about 1/4 in. to about 1 in., with most of the thickest material in the kitchen. Within the first two months, only a moderate number of curing cracks appeared. During the next two months, additional cracks appeared in all of the rooms except the kitchen. One day late in November, 1951, the kitchen floor suddenly cracked

more than any of the other areas. It was deduced that the surface had set over the thick areas of the kitchen before all of the excess moisture had escaped, and that the application of heat for several weeks, through the radiant coils, had finally caused this moisture to break through the surface.

In September, 1951, the writer built a concrete porch and steps on his home, and used mortar containing Elvacet to "touch up" the form marks. The surfaces were primed with diluted emulsion, and the thinnest possible coat of standard mix was trowelled over the surfaces, where necessary, to smooth them. In June, 1959, this mortar was slightly darker in color than the surrounding, regular concrete, but was still in place without evidence of crazing or spalling.

During the spring of 1952, the Foster Transformer Company, of Cincinnati, planned to remodel its engineering office. Its building was of timber construction with brick bearing walls, and the hardwood floor of the engineering office was very rough and almost impossible to keep clean. As part of its remodeling plan, the company decided to experiment with covering approximately 600 sq ft of the old flooring with the standard mix. In preparation for application of the mortar, the floor was swept carefully with brooms, and primed with diluted emulsion. No effort was made to nail down boards which might have been loose, nor to clean off any of the numerous patches of foreign material on the floor. After the priming, screed strips were set, and the standard-mix mortar was applied about 5/8 in. thick over about 100 sq ft. Two days later this surfacing was found to be badly cracked, with almost all of the cracks running parallel to the joints and grain of the wood. This mortar was pried off the floor—and it was found to be quite well bonded to the wood—,expanded metal lath was nailed to the old floor, and new mortar was applied. A few cracks developed in this second application, but they did not seem to be related to the grain of the wood. Six years later this floor was still giving fairly satisfactory service. Certain small areas were very badly cracked, but the majority of the floor was quite sound and smooth. Employees of the company said that the wood floor under the bad spots was so loose that they could feel it deflect when a load moved over it.

The Kosmos Cement Company ships bulk cement from its manufacturing plant to its Cincinnati distribution center in steel river barges. At the distribution center, the cement is moved from the barges to the storage silo by a conveyor belt. The cement is supplied to the belt by a "front-end loader". This loader was having so much trouble hitting the rivet heads and lapped joints of the barge deck that the company decided, in the autumn of 1952, to try surfacing the deck with the standard-mix mortar to eliminate these obstructions. No screeds were used for this job, but the mortar was trowelled over the primed steel with sufficient thickness to cover the joints and rivet heads. Six years later a company representative stated, via the telephone, that they were still well satisfied with the surfacing.

The former Ohio Military Institute[7] had about 3000 sq ft of wood flooring in its recreation rooms and a large corridor. The flooring was in poor condition, and very difficult to keep clean. This area was resurfaced with the standard mix mortar during the autumn of 1952. Expanded metal lath was nailed to the

[7] OMI closed its doors in June, 1958, and its buildings will be razed before the end of 1959.

wood flooring, and the mortar was applied 5/8 in. thick over the lath. The details of this operation are not known to the writer, but extensive cracking developed throughout the area. This cracking was not as severe, however, as that which developed in a small patches in the Foster job.

In December, 1952, the Department of Civil Engineering of the University of Cincinnati was faced with the problem of converting a 500-sq ft room, with a wood-block floor, into a soil mechanics laboratory. As an experiment, an area of approximately 2 sq ft of the block floor was covered with the standard mix. About one week after it had been laid, this mortar was pried up, essentially in one large piece. The underside was found to be quite black, indicating that the mortar had bonded to the oils and tar which had been used to treat the wood, but that these bituminous materials were not well bonded to the wood. In order to obtain the necessary bond between the mortar and the blocks, expanded metal lath was nailed to the floor. Screed strips were set, the mortar was applied about 5/8 in. thick, and the surface was scored into 5 ft x 5 ft blocks. Only one curing crack, approximately 1 ft long, developed in this floor. As a further experiment, about three weeks after the new material had been put down, the surface was sanded, by a local flooring company, using an ordinary drum-type floor sander and paper. This sanding produced a very nice, smooth surface, but took considerably more effort than had been anticipated because the hand finishing had left so many slightly-high and slightly-low places. This floor can be seen in Fig. 1. After the sanding, it was given two coats of sealer, and, in the past 6 yrs, has had no additional care beyond an occasional mopping.

In July, 1953, the Department of Civil Engineering expanded its area of activity into the main laboratory of Baldwin Hall on the campus, and decided to surface an additional 6,000 sq ft of wood-block floor with the mortar. This area was exposed to the direct rays of the sun during certain parts of the day, and contained many 10 in. x 12 in. electric outlet boxes. These two facts led to the development of a number of rather long, narrow, curing cracks, mostly radiating from the corners of the outlet boxes. In all other respects the surfacing operation was similar to that in the soils laboratory, except that this second area was not sanded. Fig. 2 is a general view of this area.

Being adjacent to each other, the last two jobs have many features in common: (1) structural concrete slabs support the old wood-block floors so that deflections are negligible. (2) Small, scattered areas of the old floors were of concrete, and the mortar bond to these areas has been entirely satisfactory. (3) No new cracks have developed since those of the initial curing. (4) The surfacing sounds quite loose and "hollow" when rapped with a hard object, or even with one's knuckles. (5) Soil compaction tests are regularly run with the molds resting on these floors, and several heavy objects have accidently been dropped on the floors, but only three very small chippings have occurred, and these were under extreme conditions. (6) The overall service ratings for these floors would be "very satisfactory".

WORK DONE BY OTHERS

To the best of the writer's knowledge, there are only three other sources of information on the use of polyvinyl acetate in cement mortars.

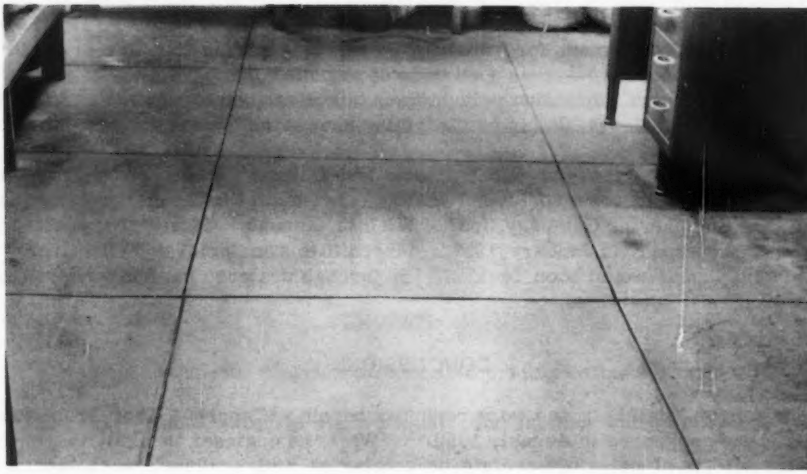

FIG. 1.—SANDED FLOOR IN SOIL MECHANICS LABORATORY AFTER SIX YEARS USE

FIG. 2.—TROWEL FINISHED FLOOR IN MAIN LABORATORY AFTER SIX YEARS USE

In 1952, Geist, Amagno, and Mellon, writing on "Improved Portland Cement Mortars with Polyvinyl Acetate Emulsion"[8], described many laboratory tests which they had conducted. While their report contains numerous tables and graphs of experimental results, it does not mention any full-scale installations.

Meyer Immerman was issued U. S. Patent 2,768,563, in October, 1956, for a "resin-bonded Cement for Repair of Concrete". According to a brief abstract,[9] this process involves a mixture of organic solvents and one of several possible polymers, including polyvinyl acetate, designed to repair cracks and pits in concrete floors. So far as the writer knows, this patent does not include the use of emulsions.

The December issue of "Contractor's and Engineer's Monthly" carried a brief announcement of a "polyvinyl acetate concentrate" being marketed by the Surface Engineering Company, Inc. of Wichita, Kansas. Literature, received from this company in January, 1957, referred to this material as "Tite-Crete", and said that it would soon be available through dealers, but the writer has never seen it advertised.

CONCLUSIONS

A popular article on the work reported herein, "Concrete That Promises Miracles", appeared in August, 1952.[10] While no engineer is likely to refer to the effects of polyvinyl acetate on mortar as "miraculous", such mortar does have three outstanding characteristics: (1) it will bond to almost any type of reasonably clean, firm surface; (2) it has high tensile strength; and (3) it cures itself in the presence of air and light without special attention. In addition, it is easy to mix and place, and has good durability as a floor surfacing material.

The perfect material still to be found, this mortar has two major disadvantages: (1) it cannot be exposed continuously to water, although periodic wetting will not affect it seriously; and (2) it cannot be applied in thicknesses much greater than 1/2 in. without the danger of serious cracking.

The writer believes that there are many floor surfacing problems which may be solved very nicely by portland cement mortar containing polyvinyl acetate emulsion.

ACKNOWLEDGMENTS

The writer wishes to acknowledge, with sincere appreciation, (1) the sponsorship and valuable suggestions of Arthur C. Avril, President of Sakrete, Inc., (2) the guidance and help of Fred O'Flaherty, Director of the University of Cincinnati Research Foundation, and (3) the constant encouragement of Cornelius Wandmacher, formerly Head of the Department of Civil Engineering, and now Associate Dean of Engineering, University of Cincinnati.

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[8] Industrial & Engineering Chemistry, v. 45, pp 59-67, April 1953.

[9] Chemical Abstracts, v. 51, col. 2247, Feb. 10, 1957.

[10] Popular Mechanics, August 1952, pp 112-113.

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CONTROL OF GROUND WATER IN EXCAVATIONS

By W. F. Swiger,[1] M. ASCE

SYNOPSIS

This paper presents a review of the basic principles and methods of controlling ground water in excavations. The factors which must be evaluated in selecting a water control system, the various methods available, their advantages and disadvantages, and basic principles of design are discussed.

INTRODUCTION

Effective control of the water encountered in excavations frequently is the deciding factor between success and failure in such work. Over the years, a number of different methods and equipment for control and removal of water have been developed by the construction industry. Sumps, possibly the oldest of all, sheeting, well points, and large diameter wells are commonly employed. Among the more exotic systems, which are used under special conditions or when difficulties are encountered with other methods, are freezing, grouting and electro-osmosis. No single system is satisfactory under all conditions. A wise selection of the method which will be most effective and most efficient under the conditions of the specific site considered will result in minimum expense, while unwise selection may result in heavy expense and possibly failure.

Among the factors which must be evaluated and considered in selecting a system are:

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 1, February, 1960.

1 Cons. Engr. Stone & Webster Engrg. Corp., Boston, Mass.

a. Soil characteristics, such as stratification, permeability of the various strata and degree of anisotropy of each stratum

b. The distance to a free water supply which will act to recharge the aquifers

c. Space limitations, such as property boundaries or interference with other structures or construction operations

d. The effects of lowering the ground water level upon adjacent structures, either because of settlement or deterioration of the piles which support them

e. The size and depth of the excavation

f. The dewatering equipment and facilities which are available

g. Time limitations

h. The methods and equipment with which the excavation will be made

A successful system must stabilize the banks of the excavation in order that slides or slumping will not interfere with operations or pose hazards to personnel or equipment. It must provide a suitable working surface with a dry bottom upon which equipment may move and on which construction operations may be carried out with a minimum of interference. It must prevent disturbance of the bottom caused by boils or piping: such disturbance may damage or destroy the capability of the soil, at and below the bottom of the excavation, to support the load of the structure and thus require piling or a more expensive type of foundation. These diverse requirements must be accomplished with minimum cost for equipment and installation, minimum operating charges, and without interference in construction operations.

Systems of ground water control may be classified in two, broad, general classes. In the first group are those methods by which the ground water level, in and adjacent to the excavation, is depressed below the bottom of the excavation by a system which collects the water as it drains from the soil and pumps it away. In the second group are methods which interpose a barrier preventing the flow of ground water into the excavation.

Well points, deep wells and sumps are the most commonly used methods of the first group. To assure bank stability, the drawdown line should be kept below the bottom and slopes of the excavation. For deposits of isotropic material, such as beach sand deposits, this can be achieved with relatively simple dewatering systems. However, where the material shows some stratification, that is, alternating strata of sand and gravel as is the case for most river deposits or glacio fluvial deposits, the permeability in a vertical direction may be only a small fraction of that in a horizontal direction. In these materials the drawdown curve is much higher than in uniform materials. Intersection of the water surface with the sides of the excavation may occur unless this factor is recognized in laying out the dewatering system. Analytical methods for determining well point or well locations are usually based upon normal well formulae which assume uniform, isotropic soils in which the coefficient of permeability is the same, both vertically and horizontally. A. Casagrande has shown[2] that anisotropic materials can be analyzed by the same methods, provided the drawdown curves and flow nets are plotted to a distorted scale in which horizontal dimensions are reduced in relation to vertical dimensions by the ratio $\sqrt{k_{max}/k_{min}}$ and the equivalent coefficient of permeability is given by $k_e = \sqrt{(k_{max})(k_{min})}$.

2 "Seepage Through Dams," by Arthur Casagrande, Journal of the New England Water Works Association, Vol. LI, No. 2, June 1937.

In stratified materials which contain one or more strata of essentially impermeable materials, such as clay, silt or very fine silty sand, there can be no vertical migration of the water. Consequently, control of ground water in the upper aquifer, as by the well point system shown on Fig. 1 will not relieve pressures in deeper lying aquifers. Drainage of aquifers which occur below the bottom of the excavation is essential to a substantial distance below the bottom of the excavation since, otherwise, pressures in these aquifers may cause boils in the bottom and possibly piping. To assure stability, it is necessary that the weight of materials above some aquifer whose top is at plane A-A exceed the hydrostatic pressure at plane A-A. For purposes of analysis, a thin impermeable membrane at A-A is assumed. Then per unit area

$$W_1 = Z\gamma$$
$$P = 62.5\,(h + Z)$$

and since the wet weight of soil is about 125 lb per cu ft and at balance

$$W_1 = P$$

from which for bare equilibrium

$$Z = h$$

in which W_1 is the downward unit pressure of soil and contained water and P is the hydrostatic pressure at given elevation. It is apparent that the rough rule of thumb may be developed that any aquifer which lies at a depth below the bottom of excavation of less than 1.3 times the maximum height of ground water above the bottom of the excavation should be drained in order to prevent excessive bottom pressures. Such drainage may require pumping of wells or well points, but frequently it may be accomplished simply by driving wells or well points to the stratum and permitting them to flow into the excavation where the water can be collected and pumped out.

Analyses of well or well point systems for use in highly stratified materials can not be based upon common well formulae for ordinary wells; rather they must be based upon artesian flow conditions. Formulae and methods of analyses developed[3] by P. T. Bennett may be used in the analyses of dewatering systems under these conditions.

The work horse of American dewatering practice is the common well point. This is a small diameter well which is jetted or driven to the desired elevation and connected to a header system, which in turn leads to a pump. It is relatively inexpensive to install. It is flexible in its operation since additional points may be added if experience indicates the necessity, and the equipment and facilities have been developed and proved over long years of experience. The equipment is readily available on a rental or purchase basis. There are a number of organizations in the country staffed with men trained in its use and operation, and capable of analyzing and developing the necessary systems.

However, well points have certain disadvantages. The maximum drawdown is limited to about 15 ft to 18 ft below the center line of the header; thus, for deeper excavations, two or even three stages of well points, headers and pumps

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[3] Discussion by P. T. Bennett of "Relief Well Systems for Dams and Levees," by W. J. Turnbull and C. I. Mausur, Transactions, ASCE, Vol. 119, 1954, p. 862.

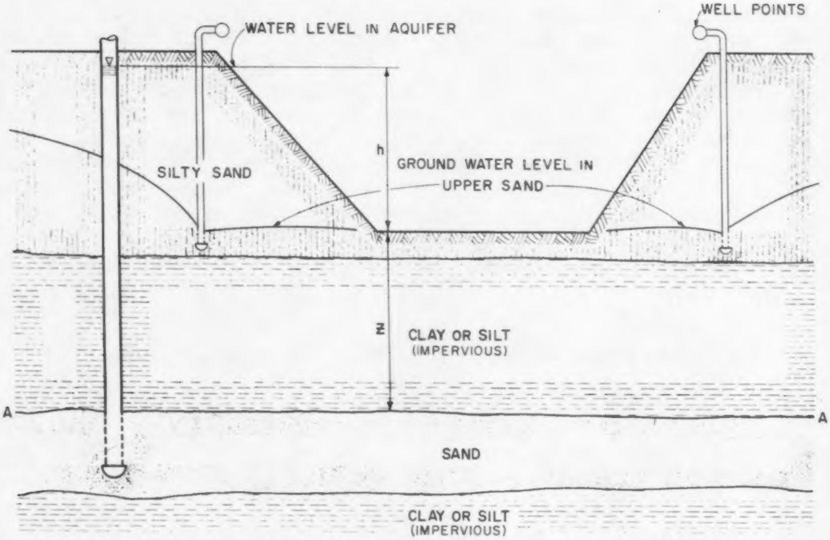

FIG. 1.—SECTION ILLUSTRATING THE DEVELOPMENT OF EXCESS HYDROSTATIC PRESSURES BELOW THE BOTTOM OF AN EXCAVATION

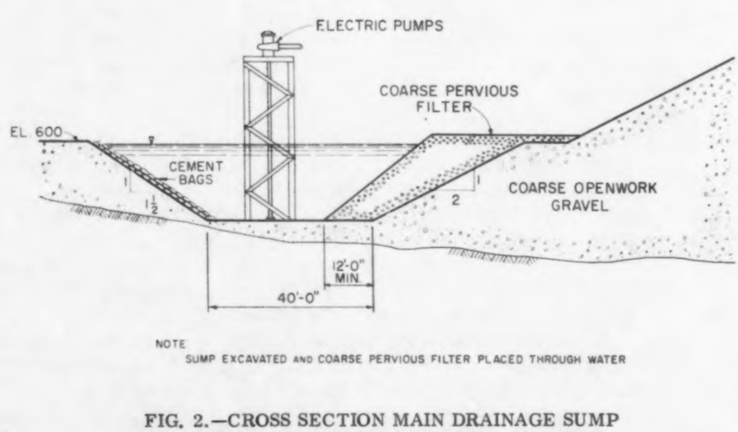

FIG. 2.—CROSS SECTION MAIN DRAINAGE SUMP ROCKY REACH HYDRO-ELECTRIC PROJECT

may be required. The berms necessary for these may significantly increase the width of the excavation, and the interruption of excavating during the times that the second and third stages of well points are installed may add to the cost and limit the types of equipment used. Entrance losses to the well points are appreciable and, since they are of small diameter, the quantity of water which can be handled by any individual point is limited. They are best adapted to sandy, or even finer, materials. In highly pervious materials, such as coarse gravels, the spacing required to handle the water may be so close that well points become impractical. In openwork materials, such as are occasionally encountered in the Columbia River area, they may not be usable. Because the well strainer is only 2 ft long, we consider the use of a sand drain around each point desirable practice in all except extremely uniform materials. A sand drain is mandatory in materials which are definitely stratified to assure that all strata within the limits of the well points are drained.

Large diameter deep wells have been used extensively in Europe and on a few jobs in the United States for dewatering. In these installations, the area to be excavated is surrounded by a number of wells driven through the various aquifers and to sufficient depth to protect against uplift pressures under the bottom of the excavation. A deep well pump is set in each, which discharges through a suitable header system. Well diameters used are commonly 6 in. to 15 in. and one job used 22 in. diameter wells. Wells may be spaced 25 ft to 100 ft or more apart, depending upon the conditions and the depth to which the water table is to be lowered. An adequate filter system through the aquifers is essential. Accordingly, the wells may be gravel packed or a gravel wall may be developed around them by surging the various aquifers.

Deep wells have the advantage that the entire system, including headers, can be spaced at some distance from the top of the excavation where it causes a minimum of interference with excavation operations. They are particularly useful for large excavations where the work will be done using earth haulers or large equipment. They have the disadvantage that the individual wells are relatively expensive and adding additional wells, in the event that the original number specified is inadequate, can be slow and difficult. Consequently, such systems require careful and detailed analysis prior to the start of construction, and driving and testing of one or more test wells may be necessary in order to arrive at an economical design.

Open sumps are possibly the oldest method of controlling ground water in excavations. As commonly built, simply by digging a small hole without providing for protection against erosion or piping or movement of the banks of the sump, they are limited to small drawdowns and modest quantities of water, and have little to recommend them. Properly designed, however, with adequate depth, size, and with the banks and bottom protected by suitable graded filters to prevent movement of the soil, they can be extremely useful, especially in dewatering very pervious material, such as openwork gravels.

Fig. 2 shows a section through the large sump used to dewater the first stage cofferdam of the Rocky Reach Hydro-Electric project. It intercepted and controlled water entering the cofferdam area through the pervious abutment on the east side of the river. The sump was excavated by drag line and the filters placed through water before the pumps were installed and the ground water lowered. This sump was approximately 30 ft deep. It was designed to collect and discharge a maximum quantity of seepage of about 160,000 gpm at a river stage of El. 650, the general floor of the excavation being El. 600. During the spring flood of 1957, it reached a maximum pumping rate of about 100,000 gpm

at a river stage of El. 638.5. Fig. 3 is a general photograph of this sump in operation. The area within the cofferdam was completely dry.

Sheeted cofferdams are commonly used where space limitations preclude the use of an open excavation, where lowering of the ground water level to permit open excavation might damage the foundations of adjoining structures, or where open water surrounds or abuts the cofferdam on one side. Typical applications include bridge piers and the intake and discharge structures for the circulating water for large steam power stations.

Where sheeting can not be driven to a cutoff on an impermeable material, such as a clay stratum or bedrock, at reasonable depth below the bottom of the

FIG. 3.—VIEW OF EAST BANK COFFERDAM OF ROCKY REACH HYDRO PROJECT DURING FLOOD OF 1957. LARGE SUMP IS AT UPPER LEFT CENTER ABOVE CRANES.

excavation, seepage under the sheeting and up into the excavation must be controlled and removed. It can be shown by a simple flow net that in uniform material a penetration of the sheeting below the bottom of the excavation equal to the height of the water above the bottom of the excavation is adequate to provide protection against piping or other disturbances of the bottom.

This is not true in stratified material and such a depth of sheeting penetration may be inadequate. A typical condition along many rivers is shown on Fig. 4. Because of the relative permeabilities of the two strata, A and B, vertical seepage flow is restricted and the pressure at point X may be almost equal to the full hydrostatic pressure at that point, the entire loss of head occurring between points X and Y. To prevent piping or boils, it is essential that the

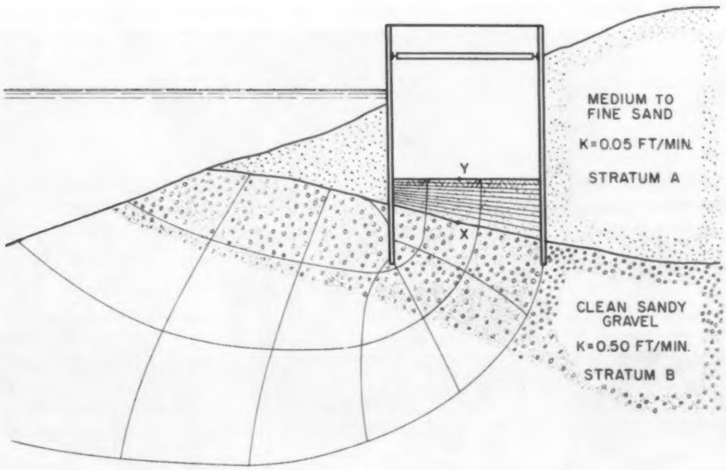

FIG. 4.—EFFECT OF VARIATIONS IN PERMEABILITY IN SHEETED COFFERDAMS

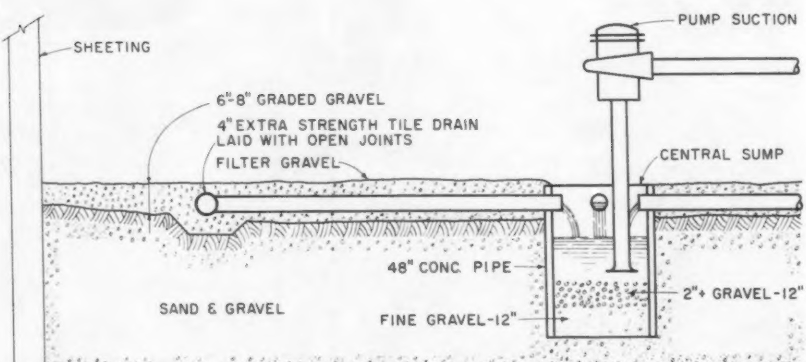

FIG. 5.—DRAIN SYSTEM IN COFFERDAM
ELRAMA STEAM STATION

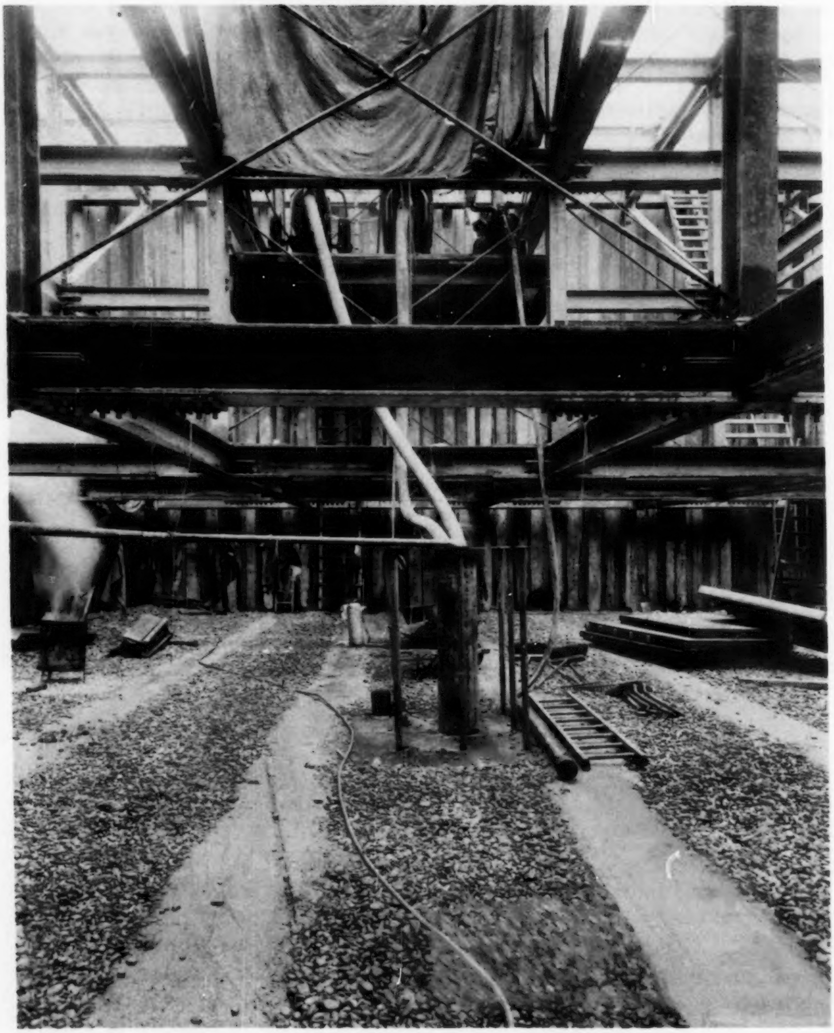

FIG. 6.—INTERIOR OF COFFERDAM, ELRAMA STEAM STATION. CENTRAL SUMP IS SHOWN AT CENTER WITH PUMPS ABOVE. TWO INCH THICK CONCRETE STRIPS HAVE BEEN PLACED IN TOP OF GRAVEL FILL TO SUPPORT CHAIRS FOR REINFORCEMENT.

sheathing be driven deeper in stratified soils, or that a suitable relief system, such as well points or wells must be provided to reduce the pressure in the aquifers underlying the bottom of the excavation. Also, in stratified soils, leakage through the interlocks may cause excessive pressures in aquifers below the bottom of the excavation and relief by drainage may be necessary.

Except when the sheeting can be driven to a cutoff, provision must be made to collect and discharge seepage which enters the excavation in such a manner as to prevent its interference with construction operations, especially concreting. This may be done by a system of well points within the cofferdam or the seepage may be collected at the bottom of the excavation. A typical collection system at the bottom of the excavation is shown on Fig. 5. This was the system used in the circulating water intake cofferdam of the Elrama Station of Duquesne Light Company, near Pittsburgh, Pa. The cofferdam was overexcavated by about 6 in. and a layer of gravel suitable graded to serve as a filter was placed over the entire bottom. An open-joint tile drain was then constructed in this gravel layer leading to a single central sump, approximate location and cross section of this drain being shown. Fig. 6 is a picture within the cofferdam during construction. The bottom of the excavation was complete-

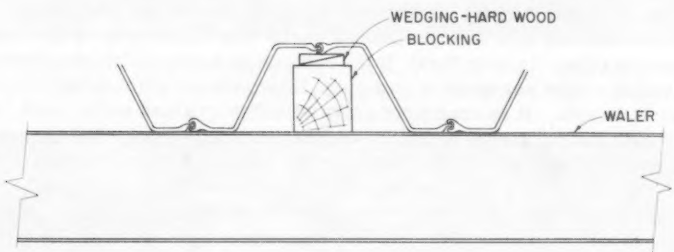

FIG. 7.—BACKING OF STEEL SHEET PILING TO PREVENT DISTORTION

ly dry and concrete was placed directly over the surface of the gravel without interference from water percolating upward through it.

The material at this site is somewhat stratified. To protect against pressures beneath the bottom of the excavation, especially in the event of high water in the river, open piezometers were installed at various locations within the excavation and instructions given the construction organization to flood the cofferdam in the event the piezometers overflowed.

Bracing of sheeted cofferdams requires careful analysis, especially when the cofferdam extends into river banks, in which case the soil loading on one side may be significantly different from that on the other. Occasionally, in order to permit the use of light sheeting and bracing, it may be found desirable to dewater around the outside of the cofferdam, using well points or a similar system. We have generally found, however, with deep section Z sheeting that the cost of dewatering systems, including their installation, maintenance and operation, exceeds the cost of the heavier bracing required if no attempt is made to dewater outside the cofferdam and the sheeting is designed for the full soil pressure and hydrostatic pressures. To prevent distortion which might cripple it, heavily loaded deep arch or Z sheeting should be blocked at each waler, as shown on Fig. 7. This blocking should be set carefully and tightly wedged to assure that it will carry loads with a minimum of movement.

Bracing systems for sheeted cofferdams may be either of wood or steel, the former being generally used only for the smaller cofferdams of limited depth. Steel walers may be field-assembled, using either welding or high strength bolts. Since sheeting can never be driven perfectly to line or to length, provision must be made for adjustment in field connections. The bracing system must provide adequate space for the operation of the excavating equipment and must be so arranged and walers so located in elevation as not to interfere with construction of the permanent structures within the cofferdam. Removal of the lower bracing systems may be necessary as the permanent structure is constructed, the load being shifted from the bracing to the permanent structure by backfilling between it and the sheeting, or by pouring the structure directly against the sheeting.

Occasionally, materials other than wood or steel are used for cofferdam bracing. Fig. 8 shows the circular cofferdam used for the second circulating water intake on the Venice No. 2 Power Station of the Union Electric Company. This cofferdam, which was 100 ft in diameter and 75 ft deep, was supported entirely by two reinforced concrete ring wales. The interior of the cofferdam was completely open, there being no cross bracing. Excavation was by means of a small dredge floating in the cofferdam. W. S. Colby has presented[4] a complete description of the design and construction of this cofferdam.

Water contained in a semipermeable mass will move toward the cathode in the presence of an electric field. If the cathode is a well point, the water which collects at it can be removed by pumping. It is on these principles that electro-osmosis is based. It is used primarily with fine grained soils, such as silts, where the electrical forces added to the gravitational seepage forces can greatly increase the rate at which water can be removed from the soil mass and thereby stabilize it. It has not been used widely in this country and the techniques of application are relatively undeveloped here. Expert advice and guidance should be employed both for analysis and design of the system if electro-osmosis is considered.

Grouting of permeable strata to effect a cutoff against ground water has been used in Europe both under dams and temporary structures, such as cofferdams for hydroelectric development and around excavations. Grouting has been used to a much lesser extent in this country. However, interest in the process is increasing and we may expect to see greater use of it, especially since new chemical grouts specifically adapted to this purpose have been developed. Several firms have entered the field and it is anticipated that continued use will result in greater experience and acceptance. Grouting materials used include cement, clay or mixtures of the two, asphaltic products and chemical grouts, such as soluble silica with a suitable reagent to cause gelling, chrome-lignin and, most recently, acrylamide monomers, such as AM-9, which polymerize after injection into the soil.

At the present time, grouting is relatively expensive and its use is generally limited to special problems or as an adjunct to other methods. It had been planned to construct the east abutment of the spillway section of the Rocky Reach Dam within a sheeted cofferdam driven to rock. Boulders, however, were encountered which prevented driving the sheeting to rock, with much of the sheeting hanging up 8 to 10 ft above the bedrock surface. Further, the rock surface was extremely irregular and seating the sheeting upon the rock surface, to form a tight joint, even when it reached, was virtually impossible.

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4 "Design and Construction of a Circulating Water Intake," by W. S. Colby, Journal of the American Concrete Institute, V. 21, No. 7, March 1959, p. 497-508.

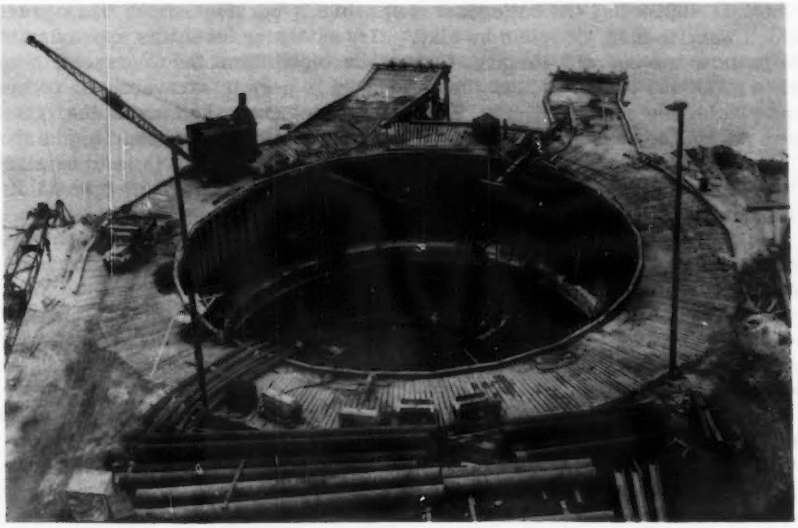

FIG. 8.—CAISSON FOR INTAKE STRUCTURE OF VENICE NO. 2 POWER PLANT. THE UPPER CONCRETE RING WALE IS IN PLACE AND THE LOWER ONE IS BEING FORMED.

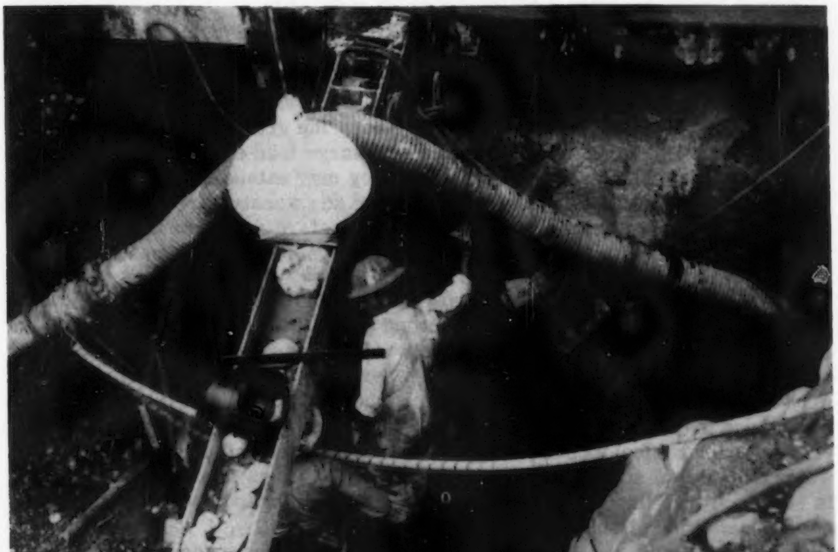

FIG. 9.—EXCAVATING FOR FOUNDATION OF EAST ABUTMENT OF ROCKY REACH PROJECT. A WALL OF GROUTED GRAVEL IS SHOWN ALONG LEFT SIDE OF PICTURE. ONE OF THE LARGE BOULDERS WHICH IMPEDED DRIVING SHEETING IS SHOWN IN THE RIGHT CENTER.

Material supporting the cofferdam was a highly pervious, open work gravel, which was, in turn, overlain by clay. Ground water level was approximately 25 ft above the bedrock surface. Under these conditions, the Contractor elected to grout the gravel around the cofferdam to permit excavating to bedrock and placing the concrete. Fig. 9 shows a photograph taken inside the cofferdam during the construction of the abutment. The tip of the sheeting is about 8 ft above bedrock surface. The wall against which concrete is to be placed is an unsupported wall of grouted gravel with approximately 20 ft to 25 ft of headwater behind it.

Freezing has been used to a limited extent in this country for control of ground water. The most recent and largest application has been for excavation of the foundation of the Gorge High Dam on the Skagit River for the City of Seattle, Department of Light and Power. Here an ice cofferdam was constructed in an arch shape to permit excavating to a depth of about 170 ft below groundwater level. The first of two of these installations has been completed and the second is being started.

There is little data available on the relative cost of the freezing processes, but it is believed this method is expensive. In the design of ice barriers, it must be kept continuously in mind that any leakage is a heat source and the greater the leakage the more heat is brought to a restricted area. Consequently, such barriers tend to be dynamically unstable. To protect against this and against irregularities in seepage characteristics of the soil, it is essential that the refrigeration system have a large margin of capacity above that theoretically necessary to freeze the material. The power source obviously must be dependable and provision of a suitable standby power source may be highly desirable if any possibility of interruption exists.

The effects of ground water lowering on adjacent structures are so well known that they need little discussion. Briefly, lowering the ground water level in a stratum of soil effectively loads it and all strata below it, since the effective stresses are increased. Thus, lowering the ground water level by 10 ft is equivalent to placing over the area a surcharge load of 625 psf. Settlements from such loads can be considerable and they may extend over wide distances where the ground water level is lowered over an extensive area by a large and long continued dewatering system. Possible effects of such ground water lowering on adjoining structures must be considered in every problem involving its use.

The second effect is possible deterioration of the foundations of adjoining structures, such as deterioration of wood piles if untreated, which would be exposed by lowering the ground water level. This is a serious problem for major construction, such as subways, where ground water levels in an area may be down for long periods of time or where drainage provided along such structures may cause a permanent lowering of the ground water level.

While discussion thus far has been directed toward the control of ground water, control of surface water adjacent to excavations is also important. It is essential that the surrounding area be properly drained; otherwise, water seeping into the soil may add appreciably to the quantity of water which must be handled in the dewatering system. Provision must be made in the dewatering system to discharge the water removed at a safe distance away from the excavation and in such a manner that it will not find its way back. To do otherwise may add considerably to the pumping costs. Severe erosion of banks may occur with resulting collection of debris and loose material in the bottom of the excavation, unless proper drainage is provided to prevent the water from

collecting and discharging down the slopes. With sheeted excavations, it is essential in the event of long or intense rains that surface water be collected and discharged away from the sheeting, since water percolating downward along the sheeting may result in developing hydrostatic pressures upon the sheeting substantially in excess of design loadings.

The broad general principles of dewatering and the factors which must be considered in any dewatering system have been summarized in this paper. Probably the most important of the several factors listed are detailed data on the character and physical properties of the soil to a substantial depth below the bottom of the excavation and evaluation of piezometric levels which may occur during the construction period in the various aquifers. Succeeding papers will discuss these matters in greater detail, including investigations, methods of analysis for various types of ground water control systems, observations on the practices and principles of installing and operating ground water control systems and several papers covering actual installations, illustrating the application of these theories and methods to practical problems.

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EPOXY RESIN FOR STRUCTURAL REPAIR OF CONCRETE PAVEMENT

By Wilson L. Davis,[1] A. M. ASCE and Eugene Pinkstaff[2]

SYNOPSIS

This paper describes tests performed in the North Central Division Laboratory of the Corps of Engineers in formulating epoxy resins that will satisfactorily repair surface defects in portland cement concrete pavements, that will bond the hardened surfaces of concrete together, and that will bond new concrete to existing concrete. It describes also the field application of the resulting epoxy resins.

INTRODUCTION

In the program of concrete runway construction in the North Central Division of the Army Engineers there has developed a need for a suitable material to make economical structural repairs of concrete pavements. Epoxy type resin was evaluated for this purpose in the laboratory and found to function satisfactorily when tested under simulated field conditions. The resin was then used to repair surface defects, to bond two adjacent pavement slabs, and to bond new concrete slabs to existing concrete pavement. The following is a description of the testing and the application of the epoxy resin for structural concrete pavement repair.

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the Construction Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. CO 1, February, 1960.

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EPOXY RESIN SYSTEM

The epoxy resin used was Epon 820.[3] This is a commercial liquid resin with an epoxide equivalent of 175-210, an average molecular weight of 350-400, and a viscosity at 25° C of 4,000-10,000 centipoises.[4] Since the epoxy resin is too brittle when cured alone for the desired use, it was flexiblized with Thiokol LP-3 (Thiokol Chemical Corp.). This is a long chain aliphatic liquid polymer containing disulfide links and reactive terminals. Thiokol LP-3 has a specific gravity of about 1.27 and a viscosity of from 7-12 poises.[5] The resin system was catalytically co-cured with DMP-10 (Rohm and Haas Co.) dimethyl amino methyl phenol) which is one of the more active tertiary amine catalysts. Its activity is such that it will effect a cure of an Epon 820-Thiokol LP-3 mixture at room temperature. Inert fillers were added, as subsequently indicated, to reduce the costs and to obtain desirable working properties. The gray color of the concrete pavement was nearly duplicated by adding titanium dioxide pigment to obtain a white opaque mixture, after which lamp black was added to obtain the desired shade of gray. For some applications the gray color would not be necessary since the cured resin would not be visible.[6]

Since the epoxy resin system cured without a flexiblizer tends to be brittle, and the addition of flexiblizer increases the percentage compressive strain, it seems obvious that for structural concrete pavement repair the amount of flexiblizer added to the epoxy resin should optimize strength versus flexibility of the cured resin system. It was therefore desirable to compare the percentage compressive strain of the resin formulations at a nominal compressive for concrete, such as 5,000 psi. The most desirable formulation was then the one that had a percentage compressive strain at 5,000 psi compressive stress that most nearly approached that of concrete, yet contained enough flexiblizer so that it did not exhibit any tendency to be brittle.

It should be noted that the number of tests were insufficient for a high degree of accuracy. The results are considered approximate but adequate. In addition, stress-strain properties of the resin are all that were tested. The remarkable resistance of the material to chemicals and water immersion has been quite well documented.[4]

The stress-strain properties of the cured resins were determined according to ASTM D 695-54, "Compressive Properties of Rigid Plastics." Test specimens were obtained by pouring the mixed resin into a cylindrical plastic mold whose dimensions were 3/4 in. by 1-1/2 in. long. After the most suitable resin system had been chosen using the criterion of optimizing strength versus flexibility, the resin was tested in the laboratory by repairing concrete beams that were broken in flexure. These tests were intended to duplicate field applications, and the results were compared with the unpatched strength of each beam. Three types of resin systems were tested, although basically they possessed much similarity. These were a liquid resin system, a mortar-type resin system, and a thixotropic-type resin system.

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[3] Shell Chemical Corp., Technical bulletins, "Epon Resins," Feb., 1958; "Epon Resins For Structural Uses," April, 1957; "Recommendations For Handling Epon Resins And Auxiliary Chemicals In Manufacturing Operations," May, 1957.

[4] "Epoxy Resins, Their Applications and Technology," by Henry Lee and Kris Neville, McGraw - Hill Book Company, New York, New York, 1957.

[5] Thiokol Chemical Corp., technical literature, "Thiokol Liquid Polymer / Epoxy Resin Casting Compounds," Sept., 1954, and "Thiokol Liquid Polymer / Epoxy Resin

[6] "Epoxy Resins," by Irving Skeist, Reinhold Publishing Corp., New York, New York, 1958.

LIQUID RESIN SYSTEM

The liquid resin system was intended to be suitable for application by pouring into vertical cracks of relatively narrow cross section compared to their depth. They would thus be inaccessible for the application of a resin patch except by this or a similar method of application. The specific problem that prompted the desire for such a material was to provide structural strength across a sawed contraction joint after a contraction crack had developed very close to the sawed joint. An example of this type of pavement failure is shown in Fig. 1. It was anticipated that in some instances one method of repair would be to patch the sawed joint and then to allow the adjacent crack to serve the function of the patched joint. This type of resin system could, of course, be adapted to repair a great variety of defective areas other than sawed contraction joints.

It was first experimentally determined that the amount of Thiokol LP-3 to be used for this purpose should be approximately 40 parts per hundred parts of Epon 820. Three mixes were then prepared in an attempt to obtain more compressive stress at a reduced compressive strain by reducing the amount of Thiokol LP-3 flexiblizer. Three mixes were prepared which contained LP-3 at 40, 35, and 30 parts per hundred of Epon 820. Otherwise they were the same. In Table 1(b) are the approximate results which were read directly from the stress-strain curves shown in Fig. 2(a).

From the standpoint of durability and impact resistance, 40 parts of Thiokol LP-3 would be the most desirable, providing its stress-strain characteristics were satisfactory for its intended use. At 30 parts of Thiokol LP-3, the cured system tends to be brittle. In both of the 30 parts compression specimens, cracking seemed evident at approximately 0.012 in. per in. compressive strain.

The proportions of the two container system for 40 parts of Thiokol LP-3 per hundred parts of Epon 820 are shown in Table 1(c).

In order to get an estimate of the amount of materials required for a particular application, it is necessary to know the number of pounds of each ingredient contained in each cubic foot of the cured resin. Since the resin cures at nearly constant volume, the quantities of the materials used constitute the quantity of the cured resin. The amount of each ingredient in 1 cu ft of the cured resin for 40 parts of Thiokol LP-3 is shown in Table 1(d).

In order to determine the usefulness of the resin system for the purpose for which it was intended, concrete flexural beams were mended with the resin in order to simulate the field conditions. Also smooth ends of two concrete beams which had been used for field flexural tests were joined together with a 1/4 in. wide patch over their entire cross section. This is shown in Fig. 3. In all five flexural beams 6 in. by 6 in. in cross section were cast with a simulated dummy joint. The mold used was 36 in. long. The flexural strength of the unpatched end was used as a direct comparison with the strength of the patched end. The simulated dummy joint was 1/4 in. wide by 1-1/2 in. deep. This was a patch of 25% of the depth of the beam. A photograph of a patched simulated dummy joint is shown in Fig. 4. Three of the patched joints were broken in simulated compression, that is, the joint was next to the applied load. Two were broken in simulated tension, that is, the joint was directly over the applied load but on the opposite side of the beam. Unfortunately the concrete was obviously of inferior quality. This did not nullify the results of the tests, however, since the strength of the patched section was compared with the strength of the unpatched section of the same beam. The results of the flexural tests are summarized in Table 2.

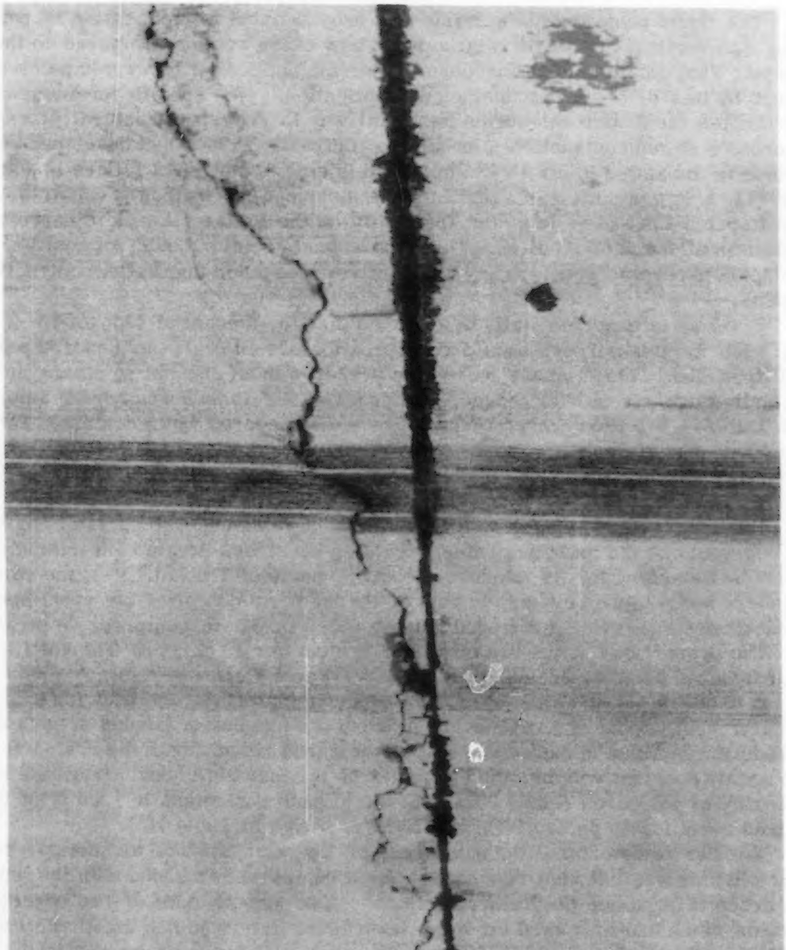

FIG. 1.—PAVEMENT FAILURE NEAR CONTRACTION JOINT

TABLE 1.—LIQUID RESIN SYSTEM

(a) Composition

A:		B:	
Epon 820	100.00 gm.	Thiokol LP-3	X gm.
Limestone filler	120.00	Titanium dioxide	25.00
		DMP-10	10.00
		Lamp black	.02

(b) Stress-Strain Properties

Grams of Thiokol LP-3	5-day air cure: Approximate peak compressive stress, in psi	Approximate % strain at peak compressive stress	Approximate % strain at compressive stress of 5,000 psi
40	8,400	6.2	3.1
35	8,370	4.6	1.2
30	8,210	2.0	0.9

(c) Proportions

A:		B:	
Epon 820	100.00 gm	Thiokol LP-3	40.00 gm
Limestone filler[a]	120.00	Titanium dioxide	25.00
	220.00 gm	DMP-10	10.00
		Lamp black	.02
			75.02 gm

$$\frac{A}{B} = \frac{220.00}{75.02} = \frac{2.9326}{1}$$

Specific gravity = 1.665 Density = 103.9 lb per cu ft

$$\text{Specific volume} = \frac{\text{ft.}^3}{103.9\text{ lb}} = 0.0096 \text{ cu ft per lb}$$

Percentage filler = 49.2%

Pot life, limit of pourability = 40 plus min

(d) Itemized Quantities

Item	Weight, in grams	Weight, in %	Weight % x density, lb per cu ft	Unit cost $/lb	Cost per cu ft of resin
Epon 820	100.00	33.90	35.22	1.58	$55.65
Titanium dioxide	25.00	8.47	8.80	0.43	3.78
Limestone filler	120.00	40.68	42.27	0.01[c]	0.42
Thiokol LP-3	40.00	13.56	14.09	1.06	14.94
DMP-10	10.00	3.39	3.52	1.82	6.41
Lamp black	.02	0.007	0.0073	1.00	0.01
Totals	295.02	100.007	103.9073		$81.21

$$\$/\text{lb} = \frac{\$81.21}{103.9\text{lb}/\text{w ft.}} = 0.782 \ \$/\text{lb}$$

[a]Commercial limestone dust such as that used in asphalt pavements. [b]Approximate small lot unit costs. The cost per cubic foot of resin will be considerably reduced in larger quantities. [c]Limestone filler is assumed to cost $0.01 per lb.

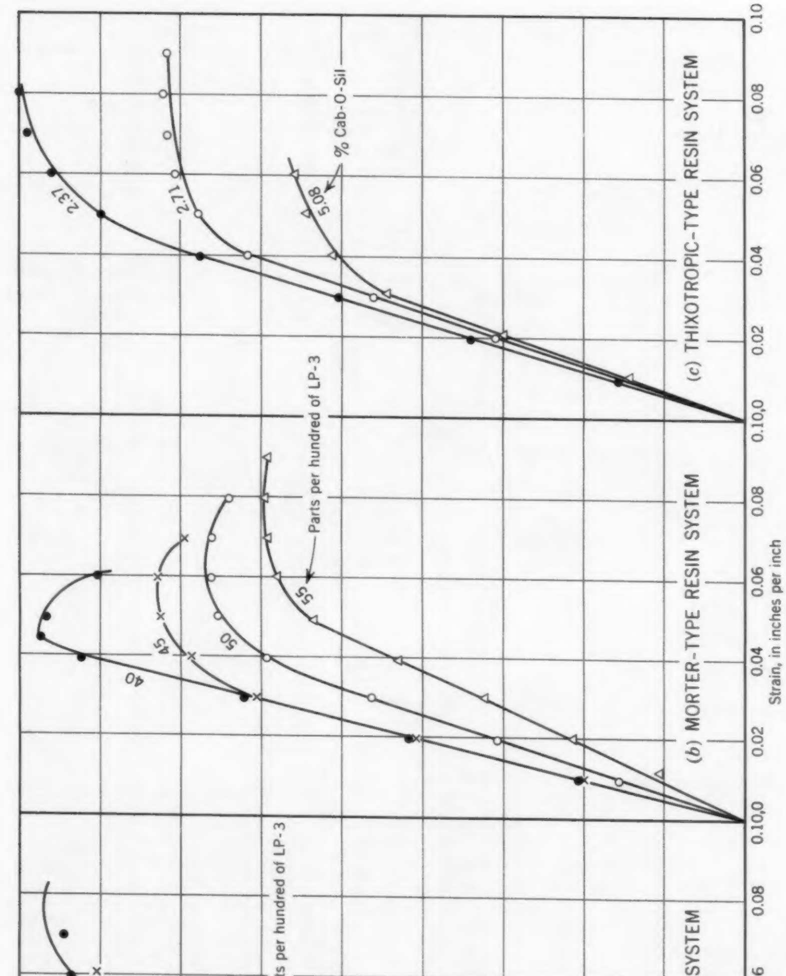
(a) LIQUID RESIN SYSTEM
(b) MORTER-TYPE RESIN SYSTEM
(c) THIXOTROPIC-TYPE RESIN SYSTEM
Parts per hundred of LP-3
Parts per hundred of LP-3
% Cab-O-Sil
30
35
40
40
45
50
55
2.37
2.71
5.08
Stress, in pounds per square inch × 10³
Strain, in inches per inch
0
0.02
0.04
0.06
0.08
0.10,0
0.10
1
2
3
4
5
6
7
8
9

FIG. 2

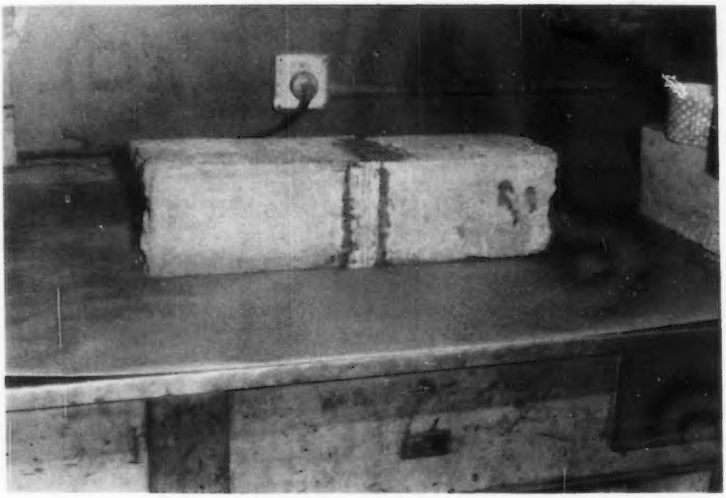

FIG. 3.—SMOOTH ENDS AFTER PATCHING

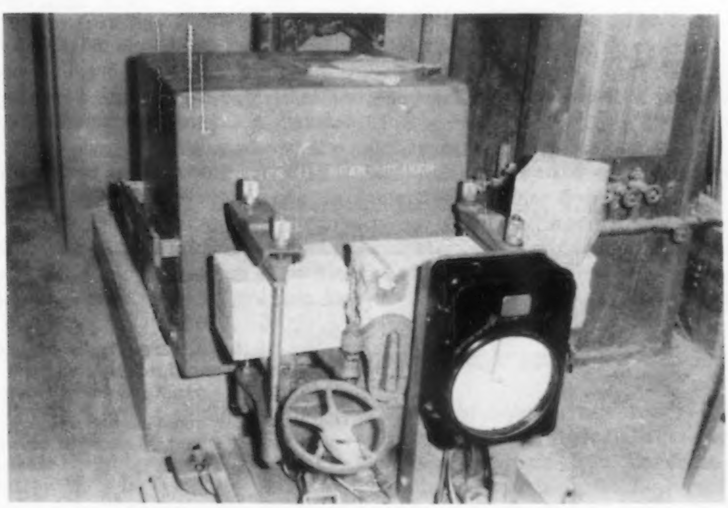

FIG. 4.—PATCHED BEAM AND BEAM BREAKER

TABLE 2.—FLEXURAL STRENGTH TESTS

Flexural strength, unpatched and, in psi	Flexural strength, patched end, in psi	Remarks[a]
(a) Liquid Resin System		
548 (Average of 4)		Age: 16 days; type III cement; 7 days moist cure
	Compression 555 (Average of 3); Tension 534 (Average of 2)	Simulated 1/4 in. by 1 1/2 in. dummy contraction joints, patched with 40 parts per hundred LP-3. Age: 15 days type III cement; 7 days moist cure. Resin: 5 days air cure.
	717	Smooth end patch 1/4 in. wide over entire cross section, patched with 40 parts per hundred LP-3. Resin: air cured 6 days.
(b) Mortar-Type Resin System		
717 (Average of 2)		Concrete: 7 days moist cure, 11 days air cure; type III cement.
	792 (Average of 2)	1/2 in. by 3 in. by 8 in. ellipsoid. Beam moist cured, 7 days and air cured 11 days. Resin: air cured 5 days.
	667 (Average of 2)	1 1/2 in. by 3 in. by 8 ellipsoid. Same cure as above.
	667	Smooth end patch 1/4 in. wide over entire cross section. Resin air cured 5 days.
(c) Thixotropic-Type Resin System		
717 (Average of 2)		Concrete: 7 days moist cure, 11 days air cure; type III cement.
	658	New concrete and resin: moist cured 5 days; new beam cast against fractured faces.
	842	New concrete and resin: moist cured 7 days; new beam cast against fractured faces.
	867	Same cure as above. Cast smooth faces.

[a]Bond failure negligible in all tests.

MORTAR-TYPE RESIN SYSTEM

The mortar-type resin system was essentially a trowelable mixture and was intended to be suitable for repairing horizontal areas of relatively large cross section compared to their depth, especially aggregate "pop outs" and spalled areas in concrete pavement. The mix was sufficiently viscous to be placed on the usual pavement grade without flowing from place.

The filler used was mortar sand conforming to ASTM C 144-52 T, "Aggregate for Masonry Mortar". Colloidal uncompressed silica was used as a thickening agent to improve the working properties and prevent settling of the sand. Titanium dioxide and lamp black were again used to obtain the desired color. Because of the mortar sand this blend is rather difficult to mix and handle.

The resin system is essentially the same as that described under the liquid resin system intended for pour application. The only difference is a filler modification which provides more desirable working properties for this type of application and also results in a cost reduction. The cured resin system was tested in compression after a 5-day air cure. The most suitable mixes tested are summarized in Table 3(a). The stress-strain properties were taken from the stress-strain curves shown in Fig. 2(b).

The mix containing 45 parts of Thiokol LP-3 was deemed the most suitable since it most nearly optimized durability and impact resistance versus stiffness and strength.

The suitability of the material for structural surface repair such as "pop outs" in concrete pavements was determined by flexural beam tests. Three 6 in. by 6 in. by 36 in. flexural test beams were cast. After the concrete had attained its initial set, areas were dug from the concrete to form surfaces that approximated the shape of ellipsoids. These areas then simulated "pop outs" that had formed in the field. These are shown in Fig. 5. Two ellipsoids were formed with dimensions of approximately 3 in. x 8 in. x 1/2 in. deep, and two were formed with dimensions of approximately 3 in. x 8 in. x 1-1/2 in. deep. They were formed 8 in. long to insure that the beam broke through the resin in the center 6-in. span. The beams were then moist cured for 7 days and air cured for 6 days before patching. The "pop out" areas were then patched with the mix containing 45 parts of Thiokol LP-3 per hundred of Epon 820. After the resin had air cured for 5 days, the patched beams were all broken with the patched area in compression, that is, with the load applied next to the patched surface. Also two smooth ends were mended together from a beam that had previously been used for field flexural tests. The results are shown in Table 2.

The proportions of the two container system containing 45 parts of Thiokol LP-3 per hundred parts of Epon 820 are shown in Table 3(c). The quantity of each of the ingredients per cubic foot of cured resin is shown in Table 3(d).

THIXOTROPIC-TYPE RESIN SYSTEM

The thixotropic resin system was tested for bonding new concrete to old concrete surfaces. It was originally tested to determine its suitability for bonding a freshly placed 10 in. concrete slab to an existing 10 in. concrete pavement. The mix was essentially of the same composition as the liquid resin system containing 40 parts of Thiokol LP-3 per hundred of Epon 820. Sufficient Cab-O-Sil thickening and thixotropic agent was substituted for the limestone filler to optimize the flow properties versus the stress-strain properties.

TABLE 3.—MORTAR-TYPE RESIN SYSTEM

(a) Composition

A:		B:	
Epon 820	100.00 gm.	Thiokol LP-3	X gm.
Mortar sand	260.00	DMP-10	10.00
Cab-O-Sil	8.00	Titanium dioxide	25.00
Lamp black	.02	Cab-O-Sil	2.00

(b) Stress-Strain Properties

Grams of Thiokol LP-3	5-day air cure		
	Approximate peak compressive stress, in psi	Approximate % strain at peak compressive stress	Approximate % strain at compressive stress of 5,000 psi
40	8,800	4.6	2.4
45	7,280	5.6	2.4
50	6,680	6.0	3.2

(c) Proportions

A:		B:	
Epon 820	100.00 gm	Thiokol LP-3	45.00 gm.
Mortar sand	260.00	DMP-10	10.00
Cab-O-Sil[a]	8.00	Titanium dioxide	25.00
Lamp black	.02	Cab-O-Sil	2.00
	368.02 gm		82.00 gm

$\frac{A}{B} = \frac{368.02 \text{ gm}}{82.00 \text{ gm}} = \frac{4.488}{1}$ Specific gravity = 1.8665 $\frac{gm}{cc}$

Density = 116.5 lbs per cuf ft

Percentage filler = 65.67%

Specific volume = $\frac{\text{cu ft}}{116.5 \text{ lb}}$ = 0.0086 cu ft per lb

Pot life, limit of workability = 50 plus min

(d) Itemized Quantities

Item	Weight, in grams	Weight, in %	Weight % x density, lb per cu ft	Unit cost \$/lb	Cost per cu ft of resin
Epon 820	100.00	22.22	25.89	1.58	\$40.91
Mortar sand	260.00	57.78	67.31	0.01	0.67
Cab-O-Sil	10.00	2.22	2.59	1.32	3.42
Lamp black	0.02	0.004	0.005	1.00	0.005
Thiokol LP-3	45.00	10.00	11.65	1.06	12.35
DMP-10	10.00	2.22	2.59	1.82	4.71
Titanium dioxide	25.00	5.56	6.48	0.43	2.79
Totals	450.02	100.004	116.51		64.86

$\$/lb = \frac{64.86 \text{ \$/w ft}}{116.5 \text{ lb/w ft}} = 0.56 \text{ \$/lb}$

[a]Uncompressed silica.

FIG. 5.—SIMULATED "POP OUTS" AFTER REPAIRING AND BREAKING

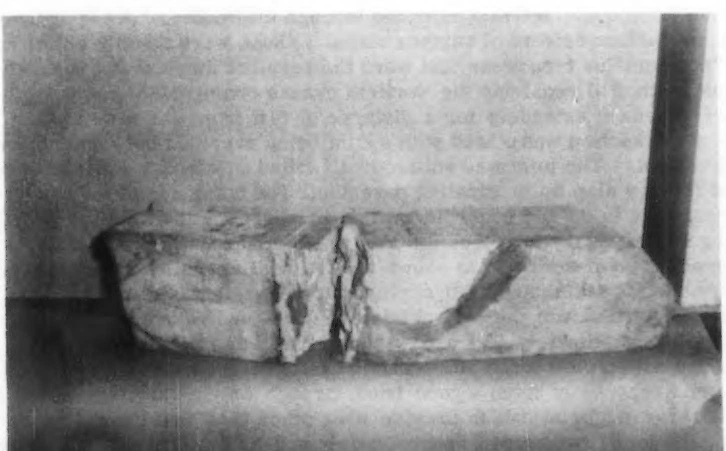

FIG. 6.—BEAM WITH BREAK BETWEEN TWO PATCHED SECTIONS

The most suitable mixes of this type tested for structural bonding between existing and freshly placed concrete pavement are summarized in Table 4(b). The stress-strain properties were taken from the graph shown in Fig. 2(c).

The mix containing 8 grams or 2.71% of the total mix of Cab-O-Sil thickening and thixotropic agent was deemed the more suitable since it had less tendency to sag and flow when spread 1/8 in. thick on a vertical concrete surface. This is shown in Fig. 7.

The suitability of the material for bonding new concrete to old concrete surfaces was tested by casting freshly mixed concrete next to hardened concrete beams that had been coated with the resin to form a layer approximately 1/8 in. thick. Two beams were cast next to old beams with fractured faces, and one beam was cast next to an old beam with a smooth surface. All the beams, both old and new, contained the same concrete mix. This was one containing 7.8 sacks of high early strength cement and 39 gal of water per cubic yard of concrete. Photographs of the flexural beams are shown in Fig. 7. The beams were patched with the mix containing 2.71% of Cab-O-Sil. The results of the flexural tests are shown in Table 2. As noted in the table the beams were moist cured until tested.

The proportions of the two container system containing 40 parts of Thiokol LP-3 per hundred of Epon 820 and with Cab-O-Sil constituting 2.71% of the total mix are shown in Table 4(c). The quantity of each of the ingredients per cubic foot of cured resin is shown in Table 4(d).

APPLICATION OF EPOXY RESIN FOR STRUCTURAL CONCRETE PAVEMENT REPAIR

The structural defects in concrete pavement consisted, for the most part, of either vertical cracks (Fig. 1), that usually occurred near a sawed contraction joint and effectively extended through the width and depth of the pavement, or surface defects of various sizes. These were usually either aggregate "pop outs" or torn areas that were the result of the joint sawing operation.

One method of repairing the vertical cracks consisted of removing a section of pavement extending for a distance of 5 ft from the sawed joint, after which a new section was placed with a joint being provided between the new and old pavement. The joint was subsequently filled with epoxy resin to bond one end of the new slab to the existing pavement. The other end of the new section was cast next to the old concrete with the fractured face of the pavement furnishing the necessary interlock for load transfer across the joint thus formed. The application of the resin is shown in Fig. 8. The resin was mixed in 35-lb batches. During this particular application the weather was wet and cool with an average temperature of approximately 40° F and an average relative humidity of approximately 95% and with intermittent precipitation. The joint was dried with a blow torch and heated slightly before pouring the resin. The resin will reportedly form a good bond through only a very thin water film. To conserve material and to provide heat other than that caused by the reaction, the joint was poured approximately one-half full of resin after which it was displaced with warm 1/2 in. to No. 4 concrete aggregate to the pavement surface. After the resin had set, the joint was troweled to grade with the thixotropic resin system, if this were necessary. The procedure was an emergency expedient and expensive. It is elaborated upon because it illustrates an unusual resin application. A better method consisted of applying the thixotopic type resin system to the existing slab before placing the new slab. The resin then cured as the concrete set.

TABLE 4.—THIXOTROPIC-TYPE RESIN SYSTEM

(a) Composition

A:		B:	
Epon 820	100.00 gm	Thiokol LP-3	40.00 gm
Cab-O-Sil	X	Titanium dioxide	25.00
Limestone filler	112.0	DMP-10	10.00
		Lamp black	.02

(b) Stress-Strain Properties

Grams of Cab-O-Sil	5-day air cure		
	Approximate peak compressive stress, in psi	Approximate % strain at peak compressive stress	Approximate % strain at compressive stress of 5,000 psi
7 (2.37% of total mix)	9,000	8.0	2.9
8 (2.71% of total mix)	7,120	7.0	3.2
15 (5.08% of total mix)	5,600	6.0	3.8

(c) Proportions

A:		B:	
Epon 820	100.00 gm	Thiokol LP-3	40.00 gm
Limestone filler	112.00	DMP-10	10.00
Cab-O-Sil	8.00	Titanium dioxide	25.00
	220.00 gm	Lamp black	0.02
			75.02 gm

$\frac{A}{B} = \frac{220.00 \text{ gm}}{75.02 \text{ gm}} = \frac{2.9326}{1}$ Specific gravity = 1.562 $\frac{\text{gm}}{\text{cc}}$

Density = 97.469 lb per cu ft

Percentage filler = 49.16%

Percentage Cab-O-Sil thixotropic agent = 2.71%

Specific volume = $\frac{\text{cu ft}}{99.467 \text{ lb}} = 0.010 \frac{\text{cu ft}}{\text{lb}}$

Pot life, limit of applicability to vertical surface = 1 1/2 plus hr

(d) Itemized Quantities

Item	Weight, in grams	Weight, in %	Weight % x density, lb per cu ft	Unit cost \$/lb	Cost per cu ft of resin
Epon 820	100.00	33.90	33.04	1.58	\$52.20
Cab-O-Sil	8.00	2.71	2.64	1.32	3.48
Limestone filler	112.00	37.97	37.00	.01	.37
Thiokol LP-3	40.00	13.56	13.22	1.06	14.01
Titanium dioxide	25.00	8.47	8.26	.43	3.55
DMP-10	10.00	3.39	3.30	1.82	6.01
Lamp black	.02	.007	.007	1.00	.007
Totals	295.02	100.007	99.467		79.63

FIG. 7.—OLD AND NEW CONCRETE THAT HAD BEEN BONDED WITH THIXOTROPIC-TYPE RESIN SYSTEM

FIG. 8.—JOINING TWO EXISTING SLABS

The surface defects, such as torn areas or areas in which aggregate "pop outs" had occurred, were repaired either by filling the area with the mortar type resin system or by filling the volume of the defective area about one-half with the liquid resin system, after which the resin was displaced to the surface of the pavement by adding a suitable concrete aggregate. Finally the repaired area was smoothed to grade, if this were necessary, with the thixotropic resin system.

The resin materials were mixed at the job site using an electric drill to which was attached a shaft and propeller. Since the necessary heat for the resin system to cure satisfactorily is provided by its own heat of reaction, the pot life, or the period of time after initial mixing that the material can be utilized, will depend on the amount mixed. For the same surrounding temperature, the larger the batch size the more heat that will be generated by the reaction and hence the shorter the pot life. The amount that can be mixed at one time will therefore depend to a great extent on the surrounding air temperature and on how soon after mixing the material is placed. The liquid system containing 40 parts of Thiokol LP-3 has been used in 35-lb batches as previously mentioned. The severity of the batch size limitation is therefore not nearly as great as might be expected. The material will cure and bond to nearly its full strength even if it has reacted to such an extent to become thick or to greatly increase in viscosity. The time limitation for its use is that it can be satisfactorily placed in the area or on the surface to be repaired.

The resin materials, especially the Epon 820 and DMP-10, should be regarded as strong skin irritants. They should be mixed in good ventilation. Neither the materials nor the fumes should be allowed to come into contact with the skin or clothing.

Cores were taken through the joints that were filled with the liquid resin and through the joints between the old and new concrete where thixotropic-type resin was used. Visual examination of these cores indicated that an excellent job had been obtained, there was no evidence in any of the cores of bond failure. In one core from a poured joint there was evidence of what must have been a loose piece of concrete that had not been removed from the edge of the old pavement. However, the resin had penetrated the crack and completely surrounded the loose particle so that in the core it was firmly bonded in place.

Four cores from the poured type of joints and two cores from the joints where new concrete was placed directly against the resin were sent to the Ohio River Division Laboratory of the Corps of Engineers for testing. Corresponding cores were taken within 5 ft of the joint at each location and these were also shipped to the laboratory for testing. All cores were subjected to the tensile splitting test as described in a paper by Sven Thaulow entitled "Tensile Splitting Test and High Strength Concrete Test Cylinders" published in the American Concrete Institute Journal, Jan., 1957. Three of the four cores from the poured joints had higher tensile splitting strength than the corresponding cores from the concrete near the joint. The fourth core had a tensile splitting strength of 70 psi less than its corresponding core. One of the two cores taken from joints where the new concrete was placed directly against the resin had a tensile splitting strength of 80 psi more than the corresponding core from concrete near the joint, while the other core had a tensile splitting strength of 50 psi less than its corresponding core.

The pavement was placed in service 7 days after being repaired. This was in the late fall. The pavement was in service during the winter months. The following spring, approximately 4 months after being repaired, an inspection

was made of the repaired areas. The repaired joints were functioning satisfactorily. The only evidence of any failure of the resin was in a few small sections that had been repaired as spalled areas. Some of these showed a bond failure along the edge of the resin patch. It is now believed that these locations were not only spalls but also included a crack located near the saw-cut and the edge of the spall. These cracks were probably functioning as active joints.

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CONTENTS

DISCUSSION

[illegible]

ENGINEERING EDUCATION AND THE CONSTRUCTION INDUSTRY: GRADUATES FOR WORK IN CONSTRUCTION[a]

Closure by David A. Day

DAVID A. DAY,[1] M. ASCE.—This paper was an attempt to analyze the educational needs of the vast construction industry and outline a general solution in terms of course areas. The treatise evidently provoked some serious thinking on the educational problems involved. Certainly much time, thought and effort will have to be expended before a completely satisfactory and workable solution will be gained. Therefore, it was encouraging and gratifying to this author to read the discussions of his paper by such prominent educators, familiar with the construction industry, as Messers, Danner, Keim and Stubbs. To think that they could agree in many respects with the original treatise was most encouraging. But it will take many more thoughtful civil engineering educators and the positive, outspoken efforts of real contributors from the industry to obtain the satisfactory, workable solution for the education of engineers for work in construction industry.

The construction industry referred to here, as in the original treatise, is the gigantic, far-reaching activity of our present day economic system that produces constructed works. This means that the industry deals with the planning, design, construction and operation, as Mr. Stubbs so aptly outlined the phases, of works permanently and solidly attached to the earth. In this broadest sense, the construction industry holds a position comparable to the manufacturing industry or the petroleum industry. Consequently, just as there are rather specific branches of engineering education providing the majority of the engineering graduates for manufacturing and petroleum work, so should it be for the construction industry. It includes the highway and airport planners and designers, the engineers who design and construct buildings and bridges, those who operate waterworks and sewage treatment plants, and many others. All of these activities are a part of the construction industry in its broadest sense and that which the U. S. Department of Commerce uses in its analyses. It is on this basis that the writer contends that a sound civil engineering curriculum should particularly benefit the construction industry. Perhaps Mr. Keim interpreted the benefit to be for construction contracting businesses only. In that case a good university educator would have to agree that the civil engineering curriculum should not be so narrowly directed, any more than it should emphasize the specialized options in the civil engineering profession.

Later, Mr. Keim makes the point that the generally accepted "curriculum should be directed at one goal, viz, a united construction industry and civil

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[a] February, 1959, by David A. Day.

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engineering profession," which does not seem to agree with the abovementioned interpretation of his previous disagreement. Based on this plea for a united effort, most civil engineering educators should agree that their curriculum must benefit the construction industry. Certainly a civil engineering curriculum would have no purpose in existing apart from a liberal arts and science program, if it did not have such a professional focal point. The thought that "degree programs at any level directed toward any particular business, industry, or profession are not education and they are not the business of a university," is far from our American concept of higher education in many respects. If this was not true, how would doctors be educated for the medical profession, or attorneys for the legal profession, or engineers for the manufacturing industry? True, the higher education, to particularly benefit these and other professions and industries, should not be in the nature of trade schools or technical institutes. There is much that universities can and should do to educate their students for both breadth and depth of knowledge and understanding while still preparing them for a professional or industrial career. This is the viewpoint that suggests a civil engineering curriculum to particularly benefit the construction industry.

Civil engineering curricula have been developing and changing with the times through a long period dating back into the 19th century. Naturally, they have emphasized what educators deemed important at that stage of development. The earliest concern was for engineered works that were designed to be technically sound. Thus, the early curricular emphasis was on design and the basic and engineering sciences leading up to it. Then, the age of specialization came into the forefront and options were introduced in each branch of engineering. The options in civil engineering were developed for the areas which suggested the greatest need for further education and specialization, such as structural engineering and sanitary engineering. The bigger state universities, with many students within a curriculum and an abundance of faculty, favored the option programs. Often the arguments for the options were that there were specialists on the large faculties and the options gave these specialists a chance to concentrate and be recognized. Also, with a large student group the association in a smaller option group gave each student a stronger sense of belonging. Of course, neither of these reasons is pedagogically sound justification for options, and it appears now that they are a thing of the past in most colleges and universities.

These earlier periods of curricular development, with emphasis on design and then on specializing options, had an adverse effect on civil engineering education for the construction industry. They overemphasized certain phases of the civil engineering profession while practically ignoring the construction phase. Such a lag in education for the construction field, as Mr. Danner suggests, is to a definite extent the fault of business and professional people in the construction field. The majority of them simply refused to acknowledge the need for men with an engineering education in their organization until after World War II.

Now the most recent stage in the development of an acceptable civil engineering curriculum is well under way. Apparently, it is prompted by a full realization that the typical civil engineering graduate will probably not work after graduation in the specialization of his choise in college, unless he does this specialization in graduate work. As a matter of fact, the new engineering graduates may work in such a variety of activities with such diversity of problems, in this age of electronics and rocketry, that many educators advo-

cate greater emphasis on basic and engineering sciences. The danger, then, is that the engineering student does not make applications and learn how to translate the scientific education into practical engineering use.

Fortunately, the predominant trend in civil engineering education apparently is toward a generalized, well-rounded cirriculum to meet the over-all needs of the construction industry in its broadest interpretation. Such a trend appeals to those many educators hopeful of a well-balanced, mature approach to civil engineering education. It builds a solid foundation of basic and engineering sciences. It includes enough design and analysis in the various important fields of civil engineering to facilitate the translation of science for practical application. And it introduces enough economics, construction methods, and management directed courses to enable the student to appreciate and understand the necessary relationships to make his original ideas become a reality. Such a well-rounded curriculum allows the student to establish some comprehension of a total project as Professor Stubbs described it. In addition, it should enable the civil engineer, who may become a design specialist, to understand construction procedures and problems that will favorably influence his design solution, as Professor Danner points out. This is the sort of generalized, well-rounded curriculum that has been adopted at the University of Denver, regardless of Mr. Keim's chiding. The more exacting education or training, beyond the fundamentals, necessary for a specialization in civil engineering must be learned after completion of the generalized curriculum suggested above. This could be gained by graduate study or, as each of the discussers suggested, by self-education beyond the formal education in a college or university.

ENGINEERING EDUCATION AND THE CONSTRUCTION INDUSTRY: WHAT THE INDUSTRY SHOULD HAVE FROM THE COLLEGES[a]

Closure by C. H. Oglesby and John W. Fondahl

C. H. OGLESBY,[1] F. ASCE and JOHN W. FONDAHL,[2] M. ASCE.—Each discusser raised questions that deserve comment:

1. Mr. Keim indicates doubt as to the value of college education beyond the fourth year for the student interested in construction. He views further study as detail work or training directed towards shortening the period until a man can pay his own way in industry.
2. Mr. Appel points out that the fundamental function of colleges is to teach and desires a more specific discussion of the type of research that might supplement this teaching effort.
3. Mr. Howard comments further on the question of governmental vs. industry sponsorship of research activity and expresses the conviction that it should be the function of the industry.

Insofar as the 4-yr curriculum is concerned, the writers agree with Mr. Keim that students should be educated as civil engineers in the broad sense. No longer is there opportunity to graduate a "construction engineer," a structural engineer, a sanitary engineer, or a hydraulics engineer in 4 yr. The already crowded curriculum does not permit any degree of specialization, and the mounting trend towards additional engineering-science courses makes it difficult to retain even the introductory professional engineering courses. The undergraduate, construction-related program at the University of California, Berkeley, described by Mr. Keim parallels that at Stanford where required courses in engineering economy and contracts and specifications plus limited opportunity for elective courses in business administration or construction equipment and methods are provided.

By implication, at least, Mr. Keim indicates his belief that a year in industry is more important to the civil engineer in construction than is a graduate year in college. The Stanford civil engineering faculty and its construction industry advisors disagree entirely with him. When Stanford embarked on a 5th year, graduate-level construction program in 1955, its aim was to give construction students the same opportunity to specialize in a field of interest that is offered to men interested in structures, hydraulics, or transportation. In view of both the complexity of the industry's problems and the primary importance of construction to our national economy, this step seemed justified. There was no intention to develop "hardware" courses or to concentrate on the details of specific construction techniques. These can be learned better

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during the apprenticeship period following graduation. Rather it was intended that, as in other fields of civil engineering, courses could be developed that would offer basic principles and approaches to problems and would develop broader viewpoints which would be of value in the long-range development of the individual and the industry itself. Moreover, it was believed that there were many courses in civil, industrial, and other branches of engineering and in the Graduate School of Business and other university departments that could contribute to a sound education for careers in construction. The degree granted at the end of 5th year of graduate study is a Master of Science in Civil Engineering. Students, if they desire, may elect to have an added subdesignation on their diplomas indicating their field of specialization such as construction, structural engineering, etc. The point is that construction is recognized as a part of civil engineering in which students can pursue advanced study on a truly professional level.

Mr. Appels' comments deal with research in the colleges and universities. The writers attempted to be specific in suggestions for types of research that might be undertaken by the engineering colleges, the reasons why the colleges were in a favorable position to conduct research, and the criteria for undertaking such work. On the other hand, some of his points call for additional comment. It is certainly true that the fundamental function of a college is to teach. However, good teaching means far more than collecting together facts about existing practices and passing them on to a new generation. Teaching at its best includes a quest for new ideas and for better ways of accomplishing goals. In other words, good teachers are several years ahead of practice, not several years behind it. Thus, in this broad sense, teaching and research are inseparable. The good teacher is continually involved in research in his field of interest. Otherwise he would stagnate and would cease to inspire his better students. Most colleges recognize the connection between good teaching and research. They encourage faculty members to participate in it, but with the stipulation that it contribute to teaching and not be an end in itself.

It is true that in some instances the balance between teaching and research has been lost. There may be faculty members who are poor teachers and good researchers or who wish to devote almost all of their time to research. In the case of tax-supported institutions, funds appropriated for teaching may be diverted to research. Likewise in private institutions money designated for teaching may at times support a faculty member's research interests. Even so, the writers can testify from their own experience that modest research efforts have enhanced their effectiveness as teachers. In their opinions it would be tragic to allow a few excesses to destroy an effective educational instrument.

Research has a second important function in education that should not be overlooked. Good students, particularly at the graduate level, are drawn primarily by the reputation of the school or department and its individual faculty members. This reputation is developed in a large measure through publications that report the results of research activity. Drawing students in this manner helps to maintain the balance between research and teaching since the students drawn to a college through the reputation of its staff members rightfully expect to take courses under these men. Research, then, makes an important contribution by bringing outstanding students.

A third value of research is to justify and support a larger and more diversified faculty. If, for example, each faculty member devotes one third time to research, a two-man faculty can be increased to three, or a four-man faculty to six. This is not to suggest, however, that a separate research

faculty be developed; to do so would destroy the usefulness of research to teaching.

A fourth function of research is to provide financial help for students who serve as part-time research assistants. Since any educational program needs good students to be successful, and many of them need financial assistance, this aspect of research can be highly important.

In summary, appropriate research activity has an important place in the college's function of teaching because:

1. It improves the quality of teaching;
2. It aids in obtaining better students; and
3. It makes possible needed financial support by offering something of value in return.

The measures of "appropriate research activity" in the colleges are:

1. The aim should be to add to basic knowledge or to develop techniques having broad application;
2. The activity must contribute to the faculty man's teaching competence;
3. The research activity must be in balance with teaching activity and subordinate to it;
4. There must be freedom to publish the results so that the findings are circulated as widely as possible; and
5. Student assistants should participate in the projects to the greatest possible degree.

If college research activities are to stay within the areas outlined above, they must avoid projects dealing with design and analysis for which the fundamental date are already available. This, in the writer's view, is consulting work, and should be treated as such. If (a) a faculty member has special knowledge or talents, and (b) he can handle a specific project and still do full justice to his obligations as a teacher, he should be free to undertake the work. If he does so, his actions should be governed by the rules of conduct for consulting engineers.

Finally there is Mr. Howard's question of governmental vs. industry sponsorship in research or construction. The writers are in full agreement with Mr. Howard that industry sponsorship is appropriate and more desirable, but probably as a long-range objective. Early prospects for such sponsorship seem poor, at least in the engineering construction field with which the writers have closest contact. Suitable projects for the colleges are generally long range and require publication of findings. Individual construction firms operate on a short-range basis and feel that they must protect any competitive advantage that they would gain by sponsoring a research project. There are, however, examples of industry-sponsored research activity in other highly competitive fields such as aircraft and electronics. Possibly these could serve as a precedent and a research program for construction could be patterned after them. The principal difference is that both of these industries are research conscious, while construction is not. Possibly, as Mr. Appel suggests, the colleges may have to educate contractors on the benefits to all of suitable research programs. This may take considerable time.

Regarding government sponsorship or research in construction: Public officials should not overlook the fact that government finally is the principal beneficiary if construction costs of public works projects are reduced. Possibly a contractor will benefit the first time the research findings are put to use, but bids will be lower on similar work in the years to come.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

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MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

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AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

c. Discussion of several papers, grouped by divisions.